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SECTION OF BITUMINOUS ROAD IN YELLOWSTONE NATIONAL PARK

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D. M. BEACH, *Editor*

Volume 21, No. 1 March 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE PHYSICAL AND CHEMICAL PROPERTIES OF PETROLEUM ASPHALTS OF THE 50-60 AND 85-100 PENETRATION GRADES[1]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Chemist, and J. Y. WELBORN, Junior Highway Engineer

FOR MANY YEARS semisolid asphalts used in the construction of the higher type bituminous pavements were produced from relatively few base petroleums by refining processes that were practically standard. More recently new sources of crude petroleums have been used and various refining methods, some of which were designed primarily to increase the yield of distillates, have been developed. Standard specifications that were thought to control adequately the quality and serviceability of these earlier asphaltic cements have been subjected to considerable questioning since the advent of these materials produced from different base petroleums and with newer refining processes. As a result, there have been changes in specification requirements by the resurrection of old tests, or the development of new tests, in an effort to obtain more satisfactory and more durable materials.

In the United States there have been three major specifications for asphalt cements around which the various State highway departments have developed their test requirements for this type of bituminous material. These are as follows:

1. Federal Specifications. These are specifications that have been adopted by the Federal Specifications Executive Committee for use by the various agencies of the Federal Government.
2. A. S. T. M. Specifications. These are specifications that were adopted as tentative some years ago by the American Society for Testing Materials but have recently been withdrawn.
3. A. A. S. H. O. Specifications. These are specifications that have been adopted by the American Association of State Highway Officials.

REQUIREMENTS OF VARIOUS SPECIFICATIONS COMPARED

The specifications for the various grades of asphalt cement in general use in road construction, as proposed or adopted by the above agencies, are given in table 1. This table includes requirements for the physical and chemical properties of the 40-50, 50-60, 60-70, 85-100, 100-120, and 120-150 penetration grades, together with their designations and the use for which they are intended. In addition to the above grades, the American Society for Testing Materials has had tentative specifications for 25-30 and 30-40 penetration asphalts, but these specifications have been omitted from table 1 since they are seldom used in road construction. The specification for the 30-40 penetration grade adopted by the American Association of State Highway Officials has likewise been omitted and for the same reason.

As shown by the last number in the grade designation, the Federal specification was initially adopted in 1925, and, after a change in the form but not in test requirements, was approved for promulgation in 1931. The A. S. T. M. specifications for 40-50, 50-60, and 60-70 penetration grades were first issued as tentative specifications in 1921 and were revised in 1922, 1923, and 1926. Specifications for the 85-100 and 100-120 grades were issued as tentative in 1921 and revised in 1922, 1923, and 1924, while that for the 120-150 grade was issued in 1922 and revised in 1923. The designations, as given in table 1, show the date of final revision for each grade followed by the letter T, to indicate a tentative standard. The term "tentative," as stated by the A. S. T. M., applies to a proposed standard published for 1 or more years with a view of eliciting criticism before it is formally adopted as standard by the Society. It is significant to note that some of these A. S. T. M. specifications remained as tentative standards for 16 years after their last revision.

Because of inability to obtain the adoption of these tentative specifications, they were withdrawn as tentative at the 1939 meeting of the A. S. T. M. The society at the present time has, therefore, no specifications for asphalt cements for use in road construction. A. A. S. H. O. specification M-20 was adopted in 1924 and revised in 1926. It is the policy of this association to adopt where possible the existing standards of the A. S. T. M. and the provisions of M-20 vary but slightly from those of the tentative standards of the A. S. T. M.

The specifications from these three sources contain general requirements that apply to the various grades as a whole. The Federal specification requires that the asphalts shall be prepared by the distillation of asphaltic petroleums. The A. A. S. H. O. specification states that the asphalts shall be prepared from petroleum while the A. S. T. M. specifications simply declare that the asphalt cement shall meet certain test requirements. All specifications require that the asphalt shall be homogeneous and free from water, and the Federal and A. A. S. H. O. specifications contain a requirement that the asphalt shall not foam when heated to 175° C. (347° F).

The Federal specification stipulates that only those asphalts that have been demonstrated by service tests as satisfactory for the intended use will be accepted. This specification also regulates the uniformity of supply of the asphalt for any given contract by controlling the specific gravity and softening point within certain limits. The A. A. S. H. O. specification states that the exact penetration grade to be used depends on the type of road, climatic conditions, and the kind and nature of the traffic; while the Federal and A. S. T. M. specifications recommend certain types of construction for each particular grade. The Federal specification also indicates the climatic conditions and amount of traffic for which each penetration grade is especially suited.

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[1] Paper presented at the meeting of the Association of Asphalt Paving Technologists, Chicago, Ill., January 30-31, 1940.

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TABLE 1.—*Summary of specifications for petroleum asphalt cements used in road construction*

Specification	Designation	Penetration grade	Intended use	Physical and chemical properties									
				Specific gravity at 77°/77° F.	Flash point	Penetration 100 gm. 5 sec. at 77° F.	Softening point	Ductility 5 cm. per minute, at 77° F.	Loss at 325° F., 5 hours	Penetration of residue as a percentage of original	CS_2 solubility		Bitumen soluble in CCl_4
											Bitumen soluble	Organic matter insoluble	
					°F.		*°F.*	*Cm.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Federal	AP-1-25 [1]	120–150	Bituminous macadam, northern United States, comparatively light traffic.	1.000+	347+	120–150	95–131	--------	1.0−	60.0+	99.5+	0.2−	--------
A. S. T. M.	D135-23T [2]	120–150	Asphalt macadam	--------	347+	120–150	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	120–150	Depends on type of road, climate, and traffic.	--------	347+	120–150	--------	30+	1.0−	60.0+	99.5+	--------	99.0+
Federal	AP-2-25 [1]	100–120	Bituminous macadam, middle United States or northern United States, comparatively heavy traffic.	1.000+	347+	100–120	95–131	--------	1.0−	60.0+	99.5+	.2−	--------
A. S. T. M.	D103-24T [2]	100–120	Asphalt macadam	--------	347+	100–120	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	100–120	Depends on type of road, climate, and traffic.	--------	347+	100–120	--------	30+	1.0−	60.0+	99.5+	--------	99.0+
Federal	AP-3-25 [1]	85–100	Bituminous macadam, southern United States.	1.000+	347+	85–100	104–140	--------	1.0−	60.0+	99.5+	.2−	--------
A. S. T. M.	D102-24T [2]	85–100	Asphalt macadam	--------	347+	85–100	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	85–100	Depends on type of road, climate, and traffic.	--------	347+	85–100	--------	30+	1.0−	60.0+	99.5+	--------	99.0+
Federal	AP-5-25 [1]	60–70	Graded bituminous concrete, northern United States, light or moderate traffic.	1.010+	347+	60–70	104–140	40+	1.0−	60.0+	99.5+	.2−	--------
A. S. T. M.	D101-26T [2]	60–70	Sheet asphalt, asphaltic concrete, asphalt macadam.	--------	347+	60–70	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	60–70	Depends on type of road, climate, and traffic.	--------	347+	60–70	--------	30+	1.0−	60.0+	99.5+	--------	99.0+
Federal	AP-6-25 [1]	50–60	Graded bituminous concrete, southern United States or northern United States, heavy traffic. Sheet asphalt, northern United States, light or moderate traffic.	1.010+	347+	50–60	104–140	40+	1.0−	60.0+	99.5+	.2−	--------
A. S. T. M.	D100-26T [2]	50–60	Sheet asphalt, asphaltic concrete	--------	347+	50–60	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	50–60	Depends on type of road, climate, and traffic.	--------	347+	50–60	--------	30+	1.0−	60.0+	99.5+	--------	99.0+
Federal	AP-7-25 [1]	40–50	Sheet asphalt, southern United States or northern United States, heavy traffic.	1.010+	347+	40–50	113–149	40+	1.0−	60.0+	99.5+	.2−	--------
A. S. T. M.	D99-26T [2]	40–50	Sheet asphalt, asphaltic concrete	--------	347+	40–50	--------	30+	2.0−	60.0+	(3)	--------	99.0+
A. A. S. H. O.	M-20 [4]	40–50	Depends on type of road, climate, and traffic.	--------	347+	40–50	--------	30+	1.0−	60.0+	99.5+	--------	99.0+

[1] Special requirements:
C-1. The materials supplied under this specification shall be asphalts prepared by the distillation of asphaltic petroleum.
C-2. Those materials only, which have been demonstrated by service tests as satisfactory for the intended use, will be acceptable under this specification.
D-1. The asphalt shall be homogeneous, free from water, and shall not foam when heated to 175° C. (347° F.).
E-2. Uniformity. The material furnished under this specification for a given contract, type, and grade shall be uniform in character and shall not vary more than 10° C. (18° F.) in softening point from the test limits specified, nor more than 0.02 in specific gravity.

[2] The asphalt shall be homogeneous and free from water.

[3] When less than 99.0 percent of the asphalt cement is soluble in carbon tetrachloride, the percentage of bitumen (soluble in carbon disulfide) shall be reported.

[4] Specification M-20 was adopted in 1924, revised in 1926. The asphalt shall be prepared from petroleum. The asphalt shall be homogeneous, free from water, and shall not foam when heated to 175° C. (347° F.).

ASPHALTS TESTED REPRESENTATIVE OF THOSE IN GENERAL USE

As shown in table 1, the physical and chemical requirements of the A. S. T. M. and A. A. S. H. O. specifications are the same except for the maximum limit of loss at 325° F. and the determination of bitumen soluble in carbon disulphide. In contrast to the A. S. T. M. and A. A. S. H. O. specifications, the Federal specification contains additional requirements for specific gravity, softening point, and organic matter insoluble in carbon disulphide, but it does not require that the solubility in carbon tetrachloride be determined. In the Federal specification the limits for the flash point and the consistency of the residue from the oven-loss test are the same as the A. S. T. M. and A. A. S. H. O. specifications, and the requirements for the percentage of loss at 325° F. and the percentage of bitumen soluble in carbon disulphide are the same as the A. A. S. H. O. specification. The Federal specification does not require a ductility test on those asphalt cements softer than the 60–70 penetration grade; and for the other grades, the ductility must be more than 40 centimeters as compared to the minimum requirement of 30 centimeters for the A. S. T. M. and A. A. S. H. O. specifications.

In order to determine how the asphalts being produced today would meet these major standard specifications, as well as other test requirements that have been used by other agencies for the control of this type of bituminous material, the Public Roads Administration requested the leading producers in the United States to submit samples of their materials corresponding to the various grades of the Federal specifications. In response to this request 30 producers submitted 42 sets of petroleum asphalt cements, of which all except one represented two or more of the penetration grades covered by the Federal specification. Several producers furnished two or more sets of asphalts differing as to the source of the base petroleum or in the method of refining. In all, 245 samples of petroleum asphalt were received, distributed among the various penetration grades as indicated in table 2.

Obviously it would take considerable time to make a detailed study of all the samples submitted. Therefore, as the asphalt cements of 50–60 and 85–100 penetration are used more generally than the other grades, and, as shown in table 2, most of the producers submitted these two grades, it was decided to confine this study more particularly to the asphalts of these grades. Twenty-seven producers furnished 39 samples of 50–60 asphalt and 28 producers furnished 40 samples of 85–100 penetration asphalt. There were two sets of asphalt samples that did not have material of either the 50–60 or 85–100 penetration grade. One of these was an asphalt from a southern California field containing a base petroleum not duplicated by any product received, and the other, an asphalt from an Oklahoma field which was duplicated in this investigation by materials of a similar type. The producer of the California asphalt submitted samples representing only the 40–50, 60–70, and 100–120 grades and the other producer did not manufacture asphalt cements harder than the 150–180 penetration grade.

Table 2.—*Classification of samples received from various producers*

Federal specification designation	Penetration grade	Total number of samples	Producers represented
	180–200	1	1
	150–180	2	2
AP-1-25	120–150	30	24
AP-2-25	100–120	33	22
AP-3-25	85–100	40	28
	70–80	4	3
AP-5-25	60–70	33	25
AP-6-25	50–60	39	27
AP-7-25	40–50	34	25
	30–40	28	20
	20–30	1	1

The source of the base petroleum and the method of refining the 50–60 and 85–100 penetration asphalt cements studied in this investigation, together with their laboratory identification and producer identification numbers, are shown in table 3. The samples of the 50–60 and 85–100 penetration asphalt of each set were produced from the same base petroleum and by the same method of refining, and they were given the same identification number. Throughout this report, these numbers, together with the designated penetration grade, will be used as the identification of the various asphalt cements.

The producers' identification numbers with the added letters A, B, or C indicate that the same producer submitted more than one set of asphalt cements which differ either as to the source of the base petroleum, the location of the refinery, or the method of refining. Producer 6 submitted samples 6, 12, and 17, representing asphalt cements refined from Colombian, Mexican, and Venezuelan petroleums, and these have been given the producers' identification numbers 6A, 6B, and 6C, respectively. Producer 7 submitted asphalts refined from Mexican and Venezuelan petroleums, while producers 13 and 24 each submitted two asphalts prepared from the same base petroleum by different refining processes. Although table 3 does not indicate any difference in the base petroleum or refining of samples 24, 25, and 26 submitted by producer 19, and samples 27 and 28 submitted by producer 20, the test results indicate differences that may be due to variations in processing. Samples 35 and 36 submitted by producer 25 were from different refineries. Samples 37 and 38 submitted by producer 26 were from the same base petroleum, but were prepared at different refineries.

ASPHALTS MET REQUIREMENTS OF A. A. S. H. O. AND A. S. T. M. SPECIFICATIONS

For all asphalts, except where noted, information as to the source of the base petroleum was furnished by the producers submitting the samples. The source of the base petroleum used in samples 27 and 28 was determined from a consideration of the producer and interpretation of the test data. The producer of samples 35 and 36 is known to refine crude petroleum from several sources; and whether these samples are products from one petroleum or blends cannot be ascertained readily from the test data only.

The principal petroleum fields that furnish the crudes for the production of asphalt cements used in the United

Table 3.—*Source and method of refining asphalt cements*

Identification No.	Producer identification	Source of base petroleum	Method of refining
1	1	California, Coalinga field	Vacuum distillation.
2	2	California, San Joaquin Valley field	Reduction and steam distillation.
3	3	do	Do.
4	4	do	Steam distillation in continuous tube still.
5	5	California, Elk Hills field	Vacuum distillation.
6	6–A	Colombia	Vacuum distillation with pipe still.
7	7–A	Mexico, Ebano field	Straight steam distillation.
8	8	Mexico	
9	9	Mexico, Panuco field	Steam distillation in Trumble (pipe) still.
10	10	do	
11	11	do	Vacuum distillation in pipe still.
12	6–B	Mexico	Do.
13	12	Venezuela	Fire and steam distillation.
14	13–A	Venezuela, Mene Grande field	Continuous distillation under sub-atmospheric pressure with steam.
15	13–B	do	Distilled in batch stills at atmospheric pressure with steam.
16	7–B	Venezuela	Straight steam distillation.
17	6–C	do	Vacuum distillation in pipe still.
18	14	do	Steam distillation.
19	15	Arkansas, Smackover field	Vacuum distillation at a low temperature.
20	16–A	do	Vacuum distillation, 89 m. p. flux.
21	16–B	do	Vacuum distillation, 101 m. p. flux.
22	17	Arkansas, Nevada County	Pipe still distillation unit and vacuum bubble tower.
23	18	Oklahoma, Cement and Walters field	
24	19–A	Oklahoma	Vacuum distillation in pipe still, partially oxidized.
25	19–B	do	Do.
26	19–C	do	Do.
27	20–A	do.[1]	
28	20–B	do	
29	21	Oklahoma, Healdton and Graham	
30	22	Kentucky and Illinois	Fire and steam distillation, possibly blown.
31	23	Mexican-duo-sol-residuum from Oklahoma crude	Steam distillation and air conversion in batch shell stills.
32	24–A	Kansas	Straight run, steam refined, vacuum process.
33	24–B	do	Produced from Winkler-Koch Shell still.
34	25–A	Wyoming	Fire and steam distillation.
35	25–B	Unknown	Do.
36	25–C	do	Do.
37	26–A	Mexico and Domestic Gulf Coast	
38	26–B	do	
39	27	Texas, Westbrook field	
40	28	Kentucky	Dubbs cracking process.

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[1] Source assumed because of the producer and from the interpretation of test results.

States are listed in table 4. As shown in table 3, all of these sources are represented in this investigation by one or more asphalts, with the exception of the Lima, Ind., the southern California, and the Trinidad fields. Petroleums from the Illinois and gulf-coastal fields are present in blends; the Illinois petroleum has been processed with Kentucky petroleum and gulf-coastal petroleum with Mexican.

The laboratory study of these selected asphalt cements included tests to show their conformity to the Federal, A. S. T. M., and A. A. S. H. O. specifications. Other tests that have been used, or are now used by various States and municipalities as specification requirements, have also been made. Additional tests which, although not usually made as control tests for asphalt cements, might develop information of value relative to their physical and chemical properties, were also made.

Where the test methods employed in this investigation have not been standardized, the essential details will be described. The A. S. T. M. and A. A. S. H. O. designations of the usual methods of test are given in table 5.

Table 4.—*Principal petroleum fields supplying asphalt cements to the United States* [1]

Country	Name of field	Area included	Type of oil
United States	Appalachian	New York, Pennsylvania, West Virginia, eastern Ohio, Kentucky, Tennessee.	Nonasphaltic.
	Lima, Ind	Michigan, northwestern Ohio, Indiana.	Do.
	Illinois	Southeastern Illinois	Semiasphaltic.
	Midcontinent	Kansas; Oklahoma; northern, western, and central Texas; northern Louisiana; southern Arkansas.	Asphaltic. Semiasphaltic. Nonasphaltic.
	Gulf-coastal	Coast of Texas and Louisiana	Asphaltic.
	Rocky Mountain	Colorado, Wyoming, Montana, Utah, and New Mexico.	Do. Semiasphaltic. Nonasphaltic.
	Northern California.	San Joaquin Valley	Asphaltic.
	Southern California.	Los Angeles Basin and coastal.	Do.
Mexico	Tampico-Tuxpan	Northern Vera Cruz (including Panuco), southern Tamaulipas.	Do.
	Tehuantepec	From southern Vera Cruz to eastern limit of Tabasco.	Do.
Venezuela	Laqunillas, La Rosa, Mene Grande.	Along eastern shore of Lake Maracaibo.	Do.
Colombia	De Barco	Region of Barranca-Bermeja	Do.
Trinidad	Trinidad	Southern portion of island	Do.

[1] Data taken from Asphalts and Allied Substances, by Abraham, 4th edition.

Table 5.—*Test methods used in the asphalt cement investigations*

Test	Method of test
Specific gravity	A. S. T. M. D 70–27.
Flash point	A. S. T. M. D 92–33.
Penetration	A. S. T. M. D 5–25.
Softening point	A. S. T. M. D 36–26.
Ductility	A. S. T. M. D 113-35.
Loss on heating	A. S. T. M. D 6–33.
Penetration of residue	A. S. T. M. D 5–25.
Saybolt Furol viscosity	A. S. T. M. D 88–36.
Float test	A. S. T. M. D 139–27.
Bitumen soluble in CS_2	A. S. T. M. D 4–27.
Bitumen soluble in CCl_4	A. S. T. M. D 165–27.
Fixed carbon	A. S. T. M. D 168–30.
Sulphur	A. S. T. M. D 129–34.
Insoluble in 86° B. naphtha	A. A. S. H. O. T–46–35.
Insoluble in ether	Do.
Oliensis test	A. A. S. H. O. T–102–38.
Film test	Kansas Highway Commission.
Toughness test	New York Department of Highways.

The results of the physical and chemical tests that show the conformity of the 50–60 and 85–100 penetration asphalts to the Federal, A. S. T. M., and A. A. S. H. O. specifications are given in tables 6 and 7. Excluding the slight lack of compliance in the penetration values, the results that fail to meet the governing specification requirement have been indicated. Seven of the asphalts of the 50–60 grade and six of the 85–100 grade did not meet one or more of the test requirements of the Federal specification. One of each grade did not meet all the test requirements specified by the A. A. S. H. O. One 50–60 penetration asphalt did not meet the requirements of the tentative A. S. T. M. specifications. In all other particulars, these materials were in substantial compliance with the specifications of these agencies.

Graphical presentations of the data tabulated in tables 6 and 7 are shown in figures 1 and 2. These figures show the range of each specification requirement and the range and average of the corresponding test values. For convenience, minimum, maximum, and average values for the laboratory tests showing conformity to the requirements of the Federal, A. S. T. M. and A. A. S. H. O. specifications have been summarized in table 8.

Since the specific gravity of an asphalt is largely dependent on the source of the base petroleum used in its manufacture, the average test value, because of the many sources represented, is of little significance. The specific gravities of the asphalts from the midcontinent fields differ widely; but the materials in this investigation which had the lowest specific gravities were from this area. Sample 30 prepared from Illinois and Kentucky petroleums also had a low gravity. The specific gravity of the Colombian asphalt, sample 6, also was low. Following, in order of increasing density as a group, were the California, Arkansas, Venezuelan, and Mexican asphalts. The materials from Texas and Wyoming also had a relatively high gravity. Samples 23, 33, and 40 were asphalts produced by cracking processes and their specific gravities were also high. The minimum requirement for specific gravity as provided for in the Federal specification would prevent the acceptance of four materials of both the 50–60 and 85–100 penetration grades.

The flash point requirement is used in specifications to enable the user to judge the fire hazard attached to the use of the asphaltic material under particular conditions of heating and applying. For the grade of asphalt cements under investigation, the minimum requirement is approximately 100° F. below the lowest flash point of any 50–60 or 85–100 penetration material and over 200° F. below the average test value for each grade. The factor of safety is, therefore, well above that required by the specifications.

ONLY TWO SAMPLES FAILED TO MEET DUCTILITY REQUIREMENTS

While a few samples had penetrations slightly higher or lower than the ranges specified for the two grades, the average test value for both the 50–60 and 85–100 penetration asphalts was approximately in the middle of the specified limits. It is believed that a satisfactory comparison of the other test characteristics of these asphalts can be made even though some of them failed to meet absolutely the penetration requirements of their particular grade.

The limits for softening point, as provided by the Federal specification, are from 104° F. to 140° F., or a range of 36° for both the 50–60 and 85–100 grades.

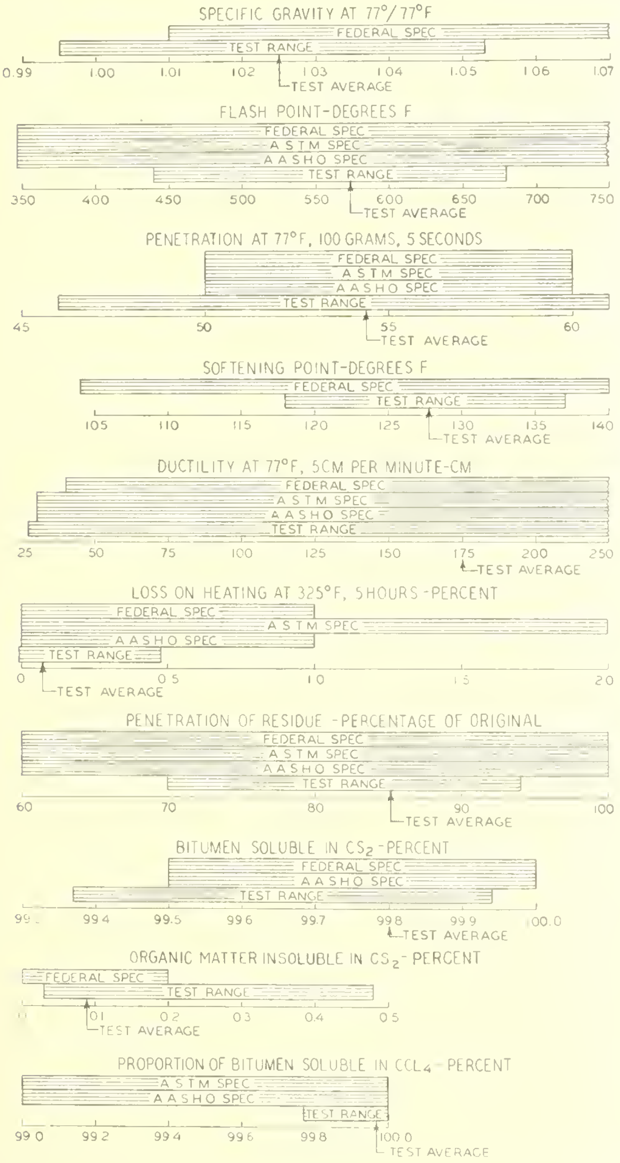

Figure 1.—Specifications and Test Data for the 50–60 Penetration Asphalts, Showing the Range of Each Specification Requirement and the Range and Average for Corresponding Laboratory Tests.

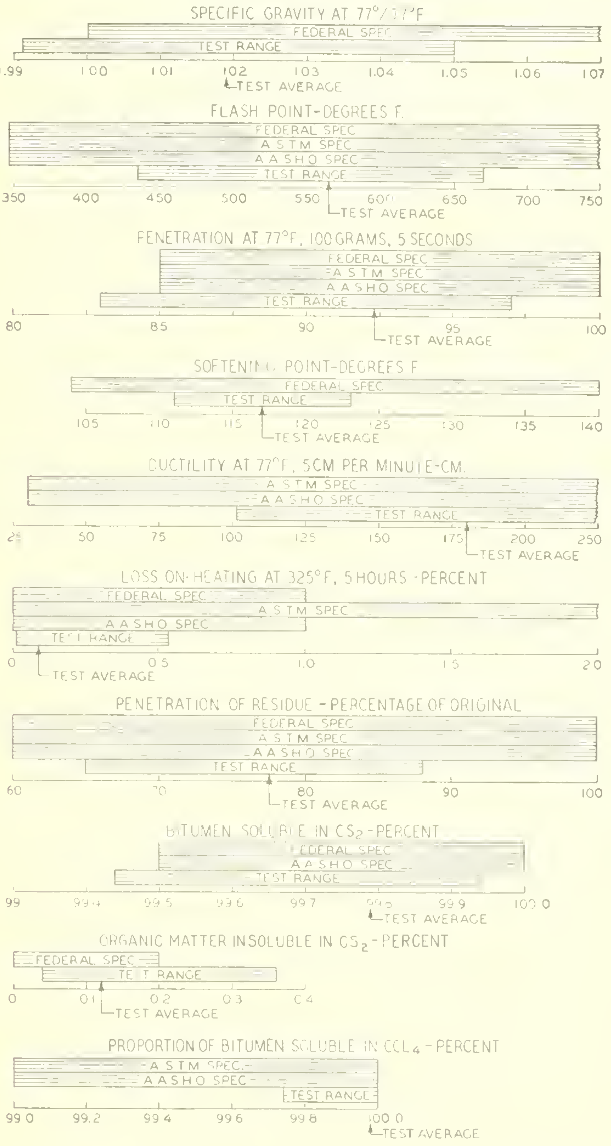

Figure 2.—Specifications and Test Data for the 85–100 Penetration Asphalts, Showing the Range of Each Specification Requirement and the Range and Average for Corresponding Laboratory Tests.

The softening points of the 50–60 asphalts varied from 118° to 137° F., a range of 19°; and the 85–100 asphalts varied from 111° to 123° F., a range of 12°. Eighteen 85–100 penetration asphalts and eight 50–60 penetration asphalts had softening points ranging from 118° to 123° F., inclusive. The average test value for each grade was approximately in the middle of the test range. While the softening point of an asphalt is dependent on the source of the base petroleum and the processing, the specification limits for the particular grade are far wider than necessary to permit the asphalts of this investigation to meet the Federal specification requirement.

The minimum ductility requirements of 40 centimeters for the Federal specification and 30 centimeters for the specifications of the A. S. T. M. and A. A. S. H. O. are very low as compared to the values obtained on the majority of asphalts tested. There were only two materials, samples 20 and 33 of the 50–60 grade, that failed to meet the requirement of the Federal specification. Sample 33 also did not meet the ductility requirement of 30 centimeters of the other two specifications. Sample 31, 50–60 grade, had a ductility very close to the minimum requirement of the Federal specification. Including these three samples, there were only eight samples of the 50–60 grade and none of the 85–100 grade that did not have a ductility greater than 100 centimeters.

Although the ductility machine used in the Public Roads Administration laboratory can measure a pull of 250 centimeters, there were many asphalts that failed to break in this distance; and these results have been reported as 250 centimeters plus. In calculating the average test values, the ductilities of 250 centimeters plus were taken as 250 centimeters. The average test values for the 50–60 and 85–100 penetration asphalts were 176 and 180 centimeters, respectively, which indicate that the ductility of the bulk of the asphalt produced is far greater than the specifications require.

Sample 40 was the only asphalt of either the 50–60

Table 6.—*Results of laboratory tests on 50–60 penetration asphalts, showing conformity to present specifications*

Identification No.	Producer number	Specific gravity at 77°/77° F.	Flash point	Penetration at 77° F., 100 gm., 5 sec.	Softening point	Ductility at 77° F., 5 cm. per minute	Loss on heating at 325° F., 50 gm., 5 hours	Penetration of residue compared to penetration before heating	CS$_2$ solubility			Bitumen soluble in CCl$_4$
									Bitumen soluble	Organic matter insoluble	Inorganic matter insoluble	
			°F.		*°F.*	*Cm.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Federal specification		1.010+	347+	50–60	104–140	40+	1.0–	60.0+	99.5+	0.2–		
A. S. T. M. specification			347+	50–60		30+	2.0–	60.0+				99.0+
A. A. S. H. O. specification			347+	50–60		30+	1.0–	60.0+	99.5+			99.0+
1	1	1.021	540	57	119	250+	0.13	84	99.88	0.03	0.09	100.00
2	2	1.013	580	61	118	250+	.05	82	99.86	.06	.08	99.96
3	3	1.013	580	61	118	250+	.05	82	99.87	.05	.08	100.00
4	4	1.016	555	60	118	250+	.06	83	99.83	.06	.11	99.96
5	5	1.016	585	58	120	250+	.07	79	99.86	.05	.09	99.98
6	6–A	1.011	630	52	126	250+	.06	88	99.87	.05	.08	99.98
7	7–A	1.041	535	58	132	197	.07	84	99.93	.04	.03	100.00
8	8	1.043	550	56	130	68	.11	82	99.87	.08	.05	99.98
9	9	1.040	545	53	132	218	.08	87	99.89	.06	.05	100.00
10	10	1.047	535	56	131	215	.12	80	99.89	.08	.03	100.00
11	11	1.045	540	54	132	180	.11	81	99.87	.06	.07	99.96
12	6–B	1.041	550	55	132	250+	.06	84	99.91	.04	.05	100.00
13	12	1.034	535	51	132	140	.11	86	99.92	.08	.00	100.00
14	13–A	1.025	600	52	126	250+	+.01	88	99.90	.09	.01	99.99
15	13–B	1.035	545	52	126	181	.02	90	99.75	.14	.11	100.00
16	7–B	1.033	560	48	132	57	.03	85	99.85	.08	.07	99.96
17	6–C	1.024	600	48	128	250+	.02	90	99.89	.08	.03	100.00
18	14	1.030	595	51	129	250+	.05	90	99.92	.06	.02	100.00
19	15	1.025	680	57	125	220	.00	86	99.83	.04	.13	99.98
20	16–A	1.017	520	58	137	[1] 36	.12	91	99.84	.08	.08	100.00
21	16–B	1.021	635	57	130	232	.02	88	99.83	.07	.10	99.97
22	17	1.017	585	57	137	96	.04	84	99.81	.06	.13	99.99
23	18	1.042	550	60	120	202	.10	70	99.72	.08	.20	99.94
24	19–A	[1] .998		54	131	84	.04	89	99.61	.18	.21	99.97
25	19–B	[1] .995	645	58	127	116	.07	84	99.67	.10	.23	99.87
26	19–C	1.017	520	53	129	78	.09	83	99.69	.15	.16	99.98
27	20–A	[1] 1.002	635	49	131	226	.03	88	99.83	.06	.11	99.92
28	20–B	1.021	525	58	126	244	.12	78	99.63	[1].21	.16	99.97
30	22	[1] 1.008	620	48	131	170	.03	94	99.79	.10	.11	100.00
31	23	1.027	495	59	133	41	.08	86	99.91	.05	.04	99.95
32	24–A	1.015	675	49	128	159	.05	90	99.69	.14	.17	99.97
33	24–B	1.037	600	46	127	[1] 27	.04	87	[1] 99.37	[1].48	.15	99.77
34	25–A	1.039	555	58	128	112	.05	79	99.94	.04	.02	99.98
35	25–B	1.014	650	57	123	219	.06	91	99.53	.04	.43	99.99
36	25–C	1.011	655	55	125	190	.07	91	99.66	.05	.29	100.00
37	26–A	1.020	525	52	132	137	.06	85	99.83	.08	.09	99.96
38	26–B	1.014	520	55	132	120	.08	84	99.81	.12	.07	100.00
39	27	1.040	620	47	129	121	.05	94	99.72	.14	.14	99.99
40	28	1.053	440	50	123	250+	.48	74	99.80	.10	.10	99.78

[1] Material fails to conform.

Table 7.—*Results of laboratory tests on 85–100 penetration asphalts, showing conformity to present specifications*

Identification No.	Producer number	Specific gravity at 77°/77° F.	Flash point	Penetration at 77° F., 100 gm., 5 sec.	Softening point	Ductility at 77° F., 5 cm. per minute	Loss on heating at 325° F., 50 gm., 5 hours	Penetration of residue compared to penetration before heating	CS$_2$ solubility			Bitumen soluble in CCl$_4$
									Bitumen soluble	Organic matter insoluble	Inorganic matter insoluble	
			°F.		*°F.*	*Cm.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Federal specification		1.000+	347+	85–100	104–140		1.0–	60.0+	99.5+	0.2–		
A. S. T. M. specification			347+	85–100		30+	2.0–	60.0+				99.0+
A. A. S. H. O. specification			347+	85–100		30+	1.0–	60.0+	99.5+			99.0+
1	1	1.019	530	85	113	223	0.16	81	99.85	0.07	0.08	100.00
2	2	1.010	540	96	111	193	.07	82	99.90	.10	.00	100.00
3	3	1.010	545	95	112	204	.07	84	99.89	.11	.00	100.00
4	4	1.013		92	112	227	.08	82	99.73	.15	.12	100.00
5	5	1.016		91	113	197	.08	81	99.81	.08	.11	100.00
6	6–A	1.004	650	92	117	185	.05	79	99.88	.12	.00	100.00
7	7–A	1.039	490	96	120	220	.10	79	99.94	.06	.00	100.00
8	8	1.038	510	96	121	102	.21	69	99.93	.07	.00	100.00
9	9	1.038	490	96	121	230	.13	78	99.86	.08	.06	100.00
10	10	1.038	485	95	121	192	.19	74	99.92	.04	.04	99.96
11	11	1.038		97	119	209	.12	77	99.91	.08	.01	100.00
12	6–B	1.039	500	97	119	242	.13	76	99.92	.05	.03	100.00
13	12	1.025	540	94	117	192	.05	74	99.84	.08	.08	99.98
14	13–A	1.020	580	95	115	192	.01	77	99.93	.06	.01	100.00
15	13–B	1.029	550	92	115	187	.04	77	99.77	.11	.12	100.00
16	7–B	1.024	565	94	117	107	.03	77	99.80	.14	.06	100.00
17	6–C	1.018	560	92	118	191	.04	78	99.82	.09	.09	100.00
18	14	1.035	500	85	123	196	.17	76	99.82	.11	.07	100.00
19	15	1.021	670	90	116	179	.02	82	99.80	.14	.06	100.00
20	16–A	1.014	500	90	121	139	.15	88	99.84	.13	.03	100.00
21	16–B	1.020	630	97	115	178	.05	80	99.74	.19	.07	100.00
22	17	1.017	585	96	117	211	.03	82	99.85	.13	.02	99.96
23	18	1.030	580	91	112	223	.04	68	99.69	.20	.11	100.00
24	19–A	[1] .995		94	118	162	.04	79	99.59	.18	23	99.97
25	19–B	[1] .991	665	94	118	152	.08	81	99.64	.19	.17	99.90
26	19–C	1.005	560	84	119	172	.08	77	99.75	.17	.08	99.95
27	20–A	[1] .999	670	93	116	164	.02	81	99.75	.14	.11	100.00
28	20–B	1.011	590	92	115	184	.06	79	99.79	.15	.06	100.00
29	21	1.031	580	92	113	200	.10	65	99.53	[1].21	.26	100.00

[1] Material fails to conform.

TABLE 7.—*Results of laboratory tests on 85–100 penetration asphalts, showing conformity to present specifications*—Continued

Identification No.	Producer number	Specific gravity at 77°/77° F.	Flash point	Penetration at 77° F., 100 gm., 5 sec.	Softening point	Ductility at 77° F., 5 cm. per minute	Loss on heating at 325° F., 50 gm., 5 hours	Penetration of residue compared to penetration before heating	CS$_2$ solubility			Bitumen soluble in CCl$_4$
									Bitumen soluble	Organic matter insoluble	Inorganic matter insoluble	
			°F.		*°F.*	*Cm.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
30	22	1.001	640	90	116	163	0.03	86	99.79	0.08	0.13	100.00
31	23	1.020	505	93	120	101	.08	75	99.88	.08	.04	100.00
32	24–A	1.010	665	85	119	173	.02	75	99.67	.11	.22	100.00
33	24–B	1.028	595	83	119	125	.04	65	[1] 99.44	[1] .36	.20	99.94
34	25–A	1.034	560	94	116	170	.02	72	99.95	.05	.00	100.00
35	25–B	1.007	560	96	116	186	.07	83	99.64	.08	.28	99.90
36	25–C	[1] .999	655	92	121	115	.03	80	99.75	.13	.12	99.97
37	26–A	1.011	530	96	118	193	.07	78	99.85	.08	.07	100.00
38	26–B	1.002	505	95	121	120	.13	80	99.82	.08	.10	100.00
39	27	1.028	590	86	116	141	.14	76	99.64	.15	.21	100.00
40	28	1.050	435	87	113	250+	.53	70	99.67	.12	.21	99.74

[1] Material fails to conform.

or 85–100 grade that had a relatively high loss on heating at 325° F. for 5 hours. The loss for both grades of this asphalt was approximately one-half the maximum allowable loss permitted by the Federal and A. A. S. H. O. specifications and one-fourth of that provided for in the A. S. T. M. specifications. The loss on heating for the two grades of sample 40 was much higher than the average test values, which are 0.07 percent for the 50–60 asphalts and 0.09 percent for the 85–100 grade. Thus it would seem that the maximum allowable loss permitted by these specifications, especially by the A. S. T. M. specifications, is far greater than necessary.

TABLE 8.—*The range and average of results of tests used as requirements in the standard specifications*

Test	50–60 grade			85–100 grade		
	Minimum	Maximum	Average	Minimum	Maximum	Average
Specific gravity at 77°/77° F.	0.995	1.053	1.025	0.991	1.050	1.019
Flash point, ° F.	440	680	574	435	670	564
Penetration at 77° F., 100 gm., 5 sec.	46	61	54.4	83	97	92.3
Softening point, ° F.	118	137	127.8	111	123	117
Ductility at 77° F., cm.	27	250+	175.5	101	250+	179.8
Loss on heating at 325° F., percent	+0.01	0.48	0.074	0.01	0.53	0.089
Penetration of residue, percentage of original	70	94	85.2	65	88	77.6
Bitumen soluble in CS$_2$, percent	99.37	99.94	99.80	99.44	99.94	99.79
Organic matter insoluble in CS$_2$, percent	0.03	0.48	0.09	0.04	0.36	0.12
Bitumen soluble in CCl$_4$, percent	99.77	100	99.97	99.74	100	99.98

FEDERAL SPECIFICATIONS FOR ASPHALT CEMENTS MOST RESTRICTIVE

The Federal, A. S. T. M. and A. A. S. H. O. specifications require that the penetration of the residue from the loss on heating shall be at least 60 percent of its original penetration before heating. The percentage of original penetration retained by the residues of the 50–60 asphalt varied from 70 to 94, with an average of 85 percent. For the 85–100 asphalts the range was from 65 to 88, with an average of 78 percent. Although all the test results were well over the minimum permitted by the specifications, there were differences among the various asphalts of each grade in the degree of hardening during the 5-hour oven test. The ranges and averages for the percentage of original penetration retained by the residues of the asphalts indicate that the 85–100 penetration asphalts harden to a greater degree than do the 50–60 penetration asphalts.

Only one 50–60 asphalt and one 85–100 penetration asphalt failed to meet the minimum requirement of the Federal and A. A. S. H. O. specifications of 99.5 percent bitumen soluble in carbon disulphide. The test average for both grades was approximately 99.8 percent. The test ranges of both the 50–60 and 85–100 penetration asphalts, as shown in figures 1 and 2, extended well beyond the limit of the Federal specification for the amount of organic matter insoluble in carbon disulphide because of the relatively high insolubility of sample 33 of each grade. All of the other materials except sample 28 of the 50–60 grade and sample 29 of the 85–100 grade met this specification requirement, and the average percentage of organic matter insoluble in carbon disulphide was approximately 0.1 for the asphalts of both penetration grades.

The minimum requirement of the A. S. T. M. and A. A. S. H. O. specifications of 99.0 percent bitumen soluble in carbon tetrachloride is far lower than any test value obtained on the asphalts of either grade. The minimum test value for each grade was approximately 99.75 percent and the average test value was 99.98 percent for the 85–100 materials and 99.97 percent for the 50–60 asphalts.

Careful study of the data that show the conformity of these asphalts to the A. S. T. M. and A. A. S. H. O. specifications furnishes no information that can be used to indicate definitely the source of the base petroleum or the refining process.

A requirement for ductility of not less than 30 centimeters as designated in these specifications may eliminate products from those petroleums that do not develop ductile residues, and this requirement may restrict to some extent the amount of blowing that can be employed in the refining process. However, with only the test data required by these specifications, it is impossible to determine the distinguishing characteristics of asphalts from various sources and refining processes. Accordingly, the uniformity of supply cannot be controlled, and no pertinent information is available to indicate what measures should be employed in the use of the particular asphalt to insure the prevention of deterioration during the mixing and construction processes and under service conditions.

The test data obtained to show compliance with the Federal specification furnish additional information that is helpful in this respect although not entirely satisfactory. The minimum specific gravity requirement serves to eliminate to some extent those asphalts prepared from petroleums of paraffinic base. The slightly higher minimum ductility requirement of this

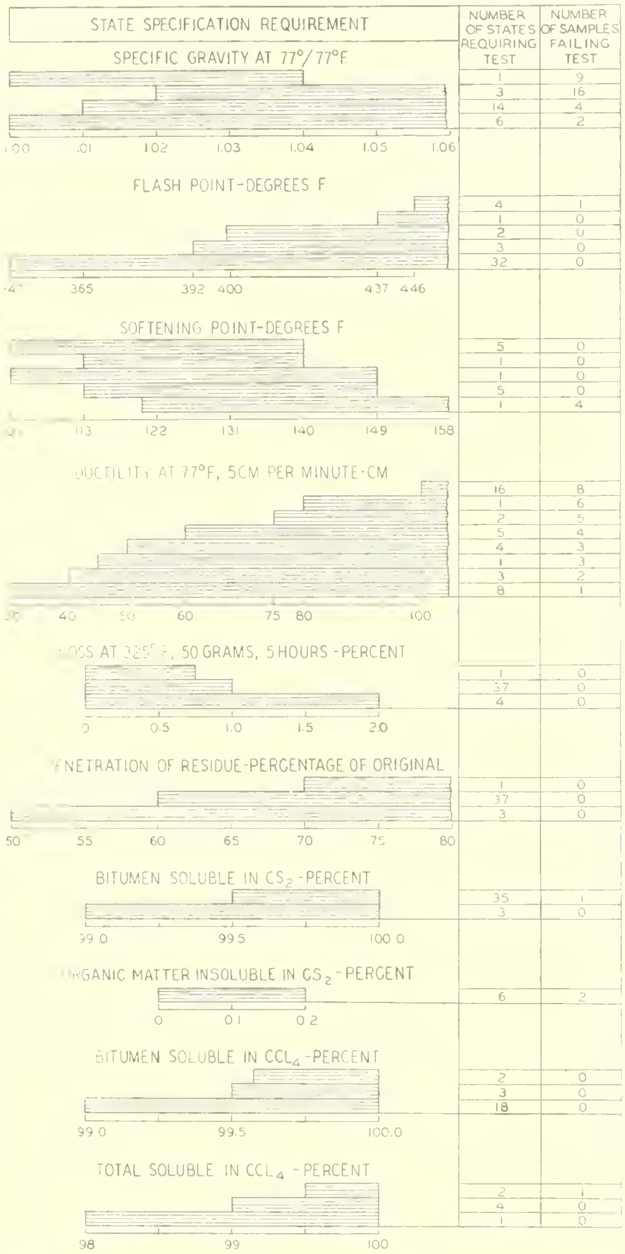

Figure 3.—State Specifications for 50–60 Penetration Asphalts, Showing the Test Requirements, the Number of States Using Each Limit and the Number of 50–60 Penetration Asphalts Investigated Failing Each Requirement.

specification, together with the softening point limits, offers some control over the processing of certain petroleums.

The uniformity of supply for a particular job is controlled in the Federal specification by the following special requirement:

> The material furnished under this specification for a given contract, type, and grade shall be uniform in character and shall not vary more than 18° F. in softening point from the test limits specified nor more than 0.020 in specific gravity * * *.

The intent of this somewhat ambiguous requirement is to provide that shipments of material for a particular job shall be within the test limits specified for the grade and in addition shall have softening points within 18° F. and specific gravities within 0.02 of values obtained for the asphalt originally accepted.

A study of the test data in tables 6 and 7 will show, however, that this provision does not insure the desired uniformity. The ranges in specific gravity values for 50–60 and 85–100 penetration asphalt were 0.995 to 1.053 and 0.991 to 1.050, respectively. If the material initially submitted had a specific gravity of 1.025, the above requirement would permit the subsequent acceptance of asphalts having specific gravities of 1.005 to 1.045. Only five 50–60 asphalts and seven 85–100 asphalts included in this investigation could not be substituted in this specific case.

The range in values of the softening point for the 50–60 asphalts was 19° F. and for the asphalts of 85–100 penetration was 12° F. It is apparent, therefore, that in both grades many of the asphalts represented by materials in this investigation could be substituted for another without violating the provisions of this special requirement for permissible variation in the softening points of subsequent shipments of asphalt for the same project and without indicating to the user that a different base petroleum or a different refining process had been substituted.

STANDARD SPECIFICATIONS CONSIDERABLY MODIFIED BY SOME STATES

The specifications of the various States for asphalt cement indicate that there is a decided tendency to modify the specifications as adopted or proposed by the Federal Specifications Executive Committee, the American Association of State Highway Officials, and the American Society for Testing Materials. A majority of the States have specifications for asphalt cements of one or more of the usual penetration grades. A study of the specifications available showed that 40 States and the District of Columbia specify 50–60 penetration asphalts. Three States do not have specifications for 50–60 but do have specifications for 60–70 penetration asphalts and these will be included in the following summary of the 50–60 grade. A majority of these specifications were adopted after 1935.

A summary of the State specifications shows that only two States use the Federal specification, two the A. A. S. H. O. specification, and one the A. S. T. M. specification, with their present test requirements. Some of the States use the same physical and chemical tests as required in one of the above-named specifications, but they have changed the limits of one or more of these requirements. Other States have changed the test limits of the Federal, A. S. T. M., or A. A. S. H. O. specifications and also have added special test requirements.

In order to show how the States have modified their specifications, the physical and chemical tests of the Federal, A. S. T. M., and A. A. S. H. O. specifications with the various test limits as adopted by the different States and the District of Columbia are shown graphically in figure 3. This figure also shows the number of States using the different limits for the specification tests and the number of asphalt cements of the 50–60 penetration grade, given in table 6, that failed to meet each specification requirement.

A discussion of the data plotted in this figure will show how the limits of the various State specifications vary from those of the Federal, A. S. T. M., or A. A. S. H. O. specifications.

Twenty-four States require a test for specific gravity at 77°/77° F. and 10 of these have test limits that differ from the requirement of the Federal specification.

Three States specify a higher minimum requirement and 6 States a lower minimum requirement. One State has both a maximum and minimum limit. In general, the higher the minimum limit, the greater the number of 50-60 asphalts that fail to meet the specification requirement. It is interesting to note that if the test requirement for specific gravity of not less than 1 or more than 1.04 were to be rigidly enforced, 5 out of 6 Mexican asphalts examined in this investigation would be excluded from use. In setting this limit it is believed that it was not the intention of this particular specification to eliminate this material.

Thirty-two of the forty-two States using the flash point test have the same requirements as the Federal, A. S. T. M., and A. A. S. H. O. specifications, while the other 10 States specify higher minimum limits ranging from 392° to 446° F. Only one 50-60 asphalt has a flash point lower than 446° F.

Only 13 States have a specification requirement for softening point and 5 of these use the same limits as the Federal specification. Of the other States, 5 use limits of 113° to 149° F., 1 uses 113° to 140° F., 1 uses 104° to 149° F., and 1 State specifies only a minimum requirement of 120° F. The 50-60 penetration asphalts meet all these limits except the last. Four of the five California asphalts tested in this investigation had softening points below 120° F.

The requirements of the States for ductility show the greatest variations from the limits of the Federal, A. S. T. M., or A. A. S. H. O. specifications. Of the 40 States requiring a ductility test at 77° F., 8 use the limit of the A. S. T. M. and A. A. S. H. O. specifications, and 3 use the limit of the Federal specification. All the remaining States have higher minimum requirements ranging from 45 centimeters up to 100 centimeters with 16 States using the highest limit. With a minimum ductility of 100 centimeters, however, only 8 of the 50-60 asphalts failed to pass this test requirement. Four asphalts, samples 16, 20, 31, and 33, had ductilities below 60 centimeters, and 3 of these, samples 20, 31, and 33, had ductilities below 45 centimeters.

Thirty-seven States have the same maximum limits for loss on heating at 325° F. as the Federal and A. A. S. H. O. specifications. Four use the A. S. T. M. requirement of not more than 2 percent and one permits a maximum loss of 0.75 percent. All of the asphalts tested had losses much lower than the 0.75 percent limit.

Thirty-seven States use the same minimum requirement of 60 for penetration of the residue from loss on heating at 325° F. expressed as the percentage of the penetration before heating. Three States specify a minimum of 50 percent and one State a minimum of 70 percent. All the asphalts met these requirements.

CERTAIN REQUIREMENTS FOUND TO BE DEFINITELY RESTRICTIVE

Thirty-five out of thirty-eight States have the same limit of 99.5 percent of bitumen soluble in carbon disulphide as the Federal and A. A. S. H. O. specifications. One sample failed to meet this requirement. Only six States require a determination of organic matter insoluble in carbon disulphide and these States use the same limit as required by the Federal specification. Two asphalts failed to meet this requirement. Twenty-three States require a determination of the proportion of bitumen soluble in carbon tetrachloride and seven States require a determination of total solubility in carbon tetrachloride. Eighteen of the twenty-three States specifying the percentage of bitumen soluble in carbon tetrachloride have the same limit as the A. S. T. M. and A. A. S. H. O. specifications. All the asphalts passed the requirements for percentage of bitumen soluble in carbon tetrachloride. Only one asphalt failed to meet the highest specification requirement for total solubility in carbon tetrachloride.

On the basis of the test data shown in table 6, seven of the 50-60 asphalts failed to pass the test requirements of the Federal specification, while but one asphalt failed to meet the test requirements of the A. S. T. M. or A. A. S. H. O. specifications. With the same control tests but with changes in the specification requirements, the limits imposed by various States, if rigidly enforced, would result in the rejection of 28 of these asphalts, leaving only 11 materials that would not be excluded by some one of the modifications. Table 9 shows the samples that failed to meet the various requirements of the States. The failure of such a large proportion of the asphalts, some of which are representative of materials which have given satisfactory service performance, indicates that certain requirements are definitely restrictive.

In addition to the routine tests that were made to determine the conformity of the 50-60 and 85-100 penetration asphalts to the Federal, A. S. T. M., and A. A. S. H. O. specifications, modifications of these routine tests and a number of other tests that have been used or are proposed for use by various agencies were also made. The data will be discussed and, where the particular test has been used as a specification requirement, the effect of the particular limits on the acceptance or rejection of materials similar to the asphalts of this study will be considered.

In table 10, the essential data for the calculation of various factors that have been used in specification requirements for controlling the susceptibility of asphalts to changes in temperature are given. Various combinations of requirements for softening point, ductility, and penetration at 32° and 115° F., together with consistency at normal temperature, 77° F., have been employed in some instances to insure a material having a satisfactory range in consistency for particular climatic conditions. In many cases various consistency tests are used to devise empirical susceptibility factors that become essential requirements in the specifications for asphaltic cements. The susceptibility factors that have been derived from the data shown in this table are given in table 11.

Since the penetration test was first adopted as a specification requirement, it has been used as a means of controlling the susceptibility of asphalts to changes in temperature. Various penetration relationships have been used for this purpose. One of the oldest methods for controlling the susceptibility of asphalt cements is the one generally used in the specifications for materials such as those intended for waterproofing and as fillers. Besides providing for a given penetration range at 77° F., these specifications require a minimum limit at 32° F., 200 grams, 60 seconds and a maximum limit at 115° F., 50 grams, 5 seconds.

SUSCEPTIBILITY FACTORS SOMETIMES ADVANCED AS SPECIFICATION REQUIREMENTS

A State specification of a few years ago employed the values obtained at these three temperatures to set up the factor:

TABLE 9.—*Identification*[1] *of the 50–60 penetration asphalts which fail to meet the test limits of the standard*[2] *specifications as modified by the States*

Test	Specific gravity at 77°/77° F.				Flash point	Softening point	Ductility at 77° F., 5 cm. per minute								Bitumen soluble in CS₂	Organic matter insoluble in CS₂	Total soluble in CCl₄
Test requirement	1.00–1.04	1.02+	1.01+	1.00+	446+ °F.	120+ °F.	100+ cm.	80+ cm.	75+ cm.	60+ cm.	50+ cm.	45+ cm.	40+ cm.	30+ cm.	99.5+ percent	0.20− percent	99.5+ percent
Sample number:																	
1						x											
2		x				x											
3		x				x											
4		x				x											
5		x															
6		x															
7	x																
8	x						x	x	x								
9																	
10	x																
11	x																
12	x																
13																	
14																	
15																	
16							x	x	x	x							
17																	
18																	
19																	
20		x					x	x	x	x	x	x	x				
21																	
22		x					x										
23	x																
24	x	x	x	x			x										
25	x	x	x	x													
26		x					x	x									
27		x	x														
28																x	
30		x	x														
31							x	x	x	x	x	x					
32		x															
33							x	x	x	x	x	x	x	x	x	x	x
34																	
35		x															
36		x															
37																	
38		x															
39																	
40	x				x												
Number of samples failing each test requirement	9	16	4	2	1	4	8	6	5	4	3	3	2	1	1	2	1

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[1] Samples marked (x) fail to meet test requirement.

[2] Includes the Federal, A. S. T. M. and A. A. S. H. O. specifications.

TABLE 10.—*Consistency determinations of the 50–60 and 85–100 penetration asphalts*

Identification No.	50–60 penetration asphalts															85–100 penetration asphalts											
	Penetration												Saybolt-Furol viscosity at 275° F.	Float test at 176° F.	Penetration										Saybolt-Furol viscosity at 275° F.	Float test at 176° F.
	200 grams, 60 seconds		100 grams, 5 seconds									50 grams, 5 seconds at 115° F.			200 grams, 60 seconds		100 grams, 5 seconds									
	At 32° F.	At 39.2° F.	At 32° F.	At 41° F.	At 50° F.	At 59° F.	At 68° F.	At 77° F.	At 86° F.	At 95° F.	At 104° F.				At 32° F.	At 39.2° F.	At 32° F.	At 41° F.	At 50° F.	At 59° F.	At 68° F.	At 77° F.	At 86° F.	At 95° F.		
													Seconds	*Seconds*											*Seconds*	*Seconds*
1	9	15	1	5	9	15	27	57	105	190	-----	--------	105	106	15	22	2	6	11	21	43	85	160	266	82	90
2	9	17	1	5	9	16	30	61	113	200	-----	--------	127	109	16	29	3	7	13	24	51	96	177	290	94	91
3	9	16	1	5	9	15	29	61	112	198	-----	--------	124	108	15	27	4	6	13	25	51	95	176	283	93	90
4	9	16	1	5	9	15	30	60	115	202	-----	--------	114	107	16	23	2	6	12	23	49	92	173	277	95	98
5	9	18	1	5	9	15	29	58	104	186	-----	--------	131	119	14	23	4	7	13	24	49	91	175	278	110	100
6	10	20	3	7	11	17	30	52	88	143	226	279	251	135	18	31	5	10	16	29	53	92	157	265	178	106
7	18	27	5	10	14	23	34	58	91	140	225	268	475	173	27	44	9	15	23	38	59	96	147	224	294	139
8	21	31	7	11	17	25	36	56	82	123	210	279	260	153	31	43	10	18	24	37	57	96	138	215	152	115
9	17	26	4	9	12	21	33	53	81	126	206	247	537	183	27	42	9	16	22	37	57	96	144	219	316	132
10	17	26	5	10	14	23	34	56	85	133	208	257	483	174	27	41	9	15	22	36	58	95	141	216	313	134
11	20	25	5	9	14	22	34	54	82	130	202	243	490	184	25	44	8	16	22	37	58	97	148	227	288	134
12	17	25	5	9	14	20	33	55	85	136	211	248	555	176	28	40	10	15	22	38	59	97	148	230	281	128
13	19	28	6	10	16	23	35	51	74	108	168	196	274	160	28	41	9	13	22	38	61	94	145	238	166	118
14	11	24	3	6	12	19	31	52	84	137	236	--------	260	131	21	35	5	10	17	32	57	95	153	258	152	109
15	15	25	4	7	13	21	33	52	84	135	225	273	274	135	22	33	5	10	17	32	57	92	152	258	147	109
16	15	25	5	8	13	21	30	48	72	106	169	205	288	160	25	37	8	13	20	37	59	94	153	250	167	113
17	14	22	5	8	13	19	30	48	78	122	196	246	337	152	25	33	6	12	19	33	55	92	151	244	197	112
18	14	26	4	7	13	21	32	51	79	123	208	248	337	147	25	37	7	11	18	33	53	85	132	214	267	135
19	12	20	5	7	12	20	36	57	96	162	265	--------	304	133	20	32	6	13	21	34	54	90	145	236	217	113
20	24	38	8	14	20	29	40	58	81	116	170	188	425	199	37	48	13	21	26	43	61	90	132	200	234	126
21	16	27	4	9	16	23	36	57	91	145	225	270	340	144	23	34	7	15	19	37	58	97	156	250	217	109
22	25	32	8	14	20	26	40	57	78	113	169	198	415	180	31	41	11	19	26	40	64	96	149	230	227	117
23	11	20	4	6	12	18	32	60	111	194	-----	--------	82	100	14	24	5	9	12	29	50	91	162	271	84	89
24	17	23	5	10	15	24	37	54	84	129	197	235	337	152	23	-----	9	14	23	37	58	94	150	230	195	103
25	14	24	5	8	16	23	35	58	96	156	248	--------	311	132	20	31	8	12	22	36	56	94	150	237	203	106
26	15	22	4	8	15	21	35	53	88	141	213	280	215	144	21	30	8	11	20	33	51	84	138	218	173	107
27	12	21	3	7	13	19	30	49	79	125	195	248	362	156	18	27	6	10	18	31	54	93	158	249	182	103
28	12	20	4	8	14	20	35	58	90	150	253	--------	196	130	15	24	5	9	14	30	51	92	157	259	148	101
29	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	--------	--------	--------	15	25	4	10	16	26	49	92	163	263	102	99

TABLE 10.—*Consistency determinations of the 50-60 and 85-100 penetration asphalts*—Continued

Identification No.	50-60 penetration asphalts														85-100 penetration asphalts											
	Penetration												Saybolt-Furol viscosity at 275° F.	Float test at 176° F.	Penetration										Saybolt-Furol viscosity at 275° F.	Float test at 176° F.
	200 grams, 60 seconds		100 grams, 5 seconds									50 grams, 5 seconds at 115° F.			200 grams, 60 seconds		100 grams, 5 seconds									
	At 32° F.	At 39.2° F.	At 32° F.	At 41° F.	At 50° F.	At 59° F.	At 68° F.	At 77° F.	At 86° F.	At 95° F.	At 104° F.				At 32° F.	At 39.2° F.	At 32° F.	At 41° F.	At 50° F.	At 59° F.	At 68° F.	At 77° F.	At 86° F.	At 95° F.		
													Seconds	*Seconds*											*Seconds*	*Seconds*
30	15	21	4	9	13	20	32	48	76	112	170	203	515	167	18	32	7	12	19	31	54	90	143	223	185	112
31	19	29	6	12	16	25	39	59	88	136	198	224	393	193	28	44	10	18	26	41	58	93	141	213	215	128
32	13	22	3	8	11	17	31	49	79	126	187	232	285	146	18	28	5	13	20	30	48	85	131	207	195	115
33	13	21	3	8	11	18	30	46	76	121	186	234	113	123	17	29	5	11	19	32	50	83	125	200	98	104
34	19	27	5	10	16	23	36	58	86	138	211	252	204	144	25	38	8	14	24	36	56	94	147	232	132	104
35	17	27	6	10	14	22	36	57	90	147	239	-------	210	129	25	40	9	16	25	38	58	96	150	248	163	109
36	16	25	4	9	13	22	36	55	87	134	212	263	254	141	26	37	10	17	27	39	60	92	136	200	172	109
37	19	29	5	10	16	24	33	52	77	116	174	206	369	180	32	47	9	17	28	42	63	96	143	223	195	122
38	21	33	7	11	17	25	37	55	81	120	176	204	324	173	42	49	14	21	29	43	65	95	138	205	195	135
39	8	14	2	6	8	13	26	47	82	134	227	276	209	139	15	23	4	9	16	25	45	86	159	265	132	103
40	13	21	3	7	12	19	31	50	82	136	220	-------	120	129	18	36	6	10	18	31	52	87	151	259	76	96

TABLE 11.—*Susceptibility factors of the 50-60 and 85-100 penetration asphalts*

Identification No.	Pen. 77° F. / Pen. 32° F. (1)		Pen. 115° F.—Pen. 32° F. / Pen. 77° F. (1) (2)	Pen. 100° F. / Pen. 77° F. (1)		Pen. 39.2° F. / Pen. 77° F. (1)		PTS[3] calculated by Pfeiffer and Van Doormal method		Slope of the log penetration-temperature curve		Penetration index				Float test index		Fluidity factor	
												Calculated from PTS[3]		Calculated from slope of log penetration temperature curve					
	50-60 grade	85-100 grade	50-60 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade	50-60 grade	85-100 grade
						Percent	*Percent*												
1	6.3	5.7	(4)	4.7	5.3	26	26	0.0276	0.0272	0.0295	0.0323	−1.39	−1.29	−1.79	−2.32	77.7	87.4	27.4	−2.6
2	6.8	6.0	(4)	4.8	5.1	28	30	.0270	.0268	.0307	.0314	−1.25	−1.20	−2.02	−2.16	81.6	93.4	40.3	−1.9
3	6.8	6.3	(4)	4.8	5.4	26	28	.0271	.0268	.0297	.0324	−1.28	−1.20	−1.83	−2.34	81.2	92.5	38.4	−1.9
4	6.7	5.8	(4)	4.9	5.2	27	25	.0272	.0272	.0307	.0315	−1.29	−1.29	−2.02	−2.18	80.2	95.0	32.4	−2.8
5	6.4	6.5	(4)	4.4	5.3	31	25	.0266	.0262	.0287	.0313	−1.15	−1.07	−1.63	−2.14	83.1	95.4	42.3	8.2
6	5.2	5.1	5.2	3.7	4.1	38	34	.0242	.0237	.0252	.0267	−.57	−.44	−.83	−1.19	83.8	98 8	103.5	79.1
7	3.2	3.6	4.3	3.2	3.1	47	46	.0209	.0213	.0217	.0219	+.42	+.25	+.17	+.10	100.1	115.5	241.9	190.1
8	2.7	3.1	4.6	2.8	2.8	55	45	.0209	.0207	.0192	.0211	+.42	+.47	+.99	+.32	92.6	105.1	114.2	53.8
9	3.1	3.6	4.3	3.2	2.9	49	44	.0216	.0211	.0226	.0217	+.20	+.32	−.12	+.15	98.5	112.5	256.5	211.2
10	3.3	3.5	4.3	3.0	3.1	46	43	.0208	.0210	.0216	.0220	+.43	+.38	+.19	+.07	98.7	112.8	239.1	207.1
11	2.7	3.9	4.1	3.1	3.1	46	45	.0212	.0217	.0216	.0219	+.33	+.17	+.19	+.10	99.6	114 0	235.4	185.3
12	3.2	3.5	4.2	3.2	3.1	45	41	.0210	.0218	.0222	.0219	+.38	+.14	-------	+.10	98.4	111.5	275.0	178.5
13	2.7	3.4	3.5	2.7	3.2	55	44	.0219	.0231	.0185	.0221	+.08	−.24	+1.26	+.03	90.4	105.3	113.7	67.7
14	4.7	4.5	(4)	3.5	3.7	46	37	.0242	.0243	.0238	.0248	−.57	−.58	−.45	−.71	82.6	101.7	108.2	54.2
15	3.5	4.2	5.0	3.3	3.8	48	36	.0244	.0246	.0224	.0248	−.63	−.67	−.05	−.71	83.8	100.1	115.4	50.6
16	3.2	3.8	4.0	2.9	3.5	52	39	.0221	.0235	.0202	.0236	+.05	−.37	+.64	−.40	87.6	103.1	115.2	68.6
17	3.4	3.7	4.8	3.3	3.7	46	36	.0241	.0229	.0224	.0246	−.52	−.20	−.05	−.67	85.4	101.5	138.7	96.6
18	3.6	3.4	4.6	3.1	3.3	51	44	.0228	.0202	.0215	.0226	−.18	+.68	+.22	−.12	86.6	107.0	145.9	154.7
19	4.8	4.5	(4)	3.7	3.4	35	36	.0238	.0246	.0249	.0235	−.45	−.67	−.74	−.37	87.0	100.8	140.8	114.3
20	2.4	2.4	2.8	2.5	2.7	66	53	.0189	.0217	.0172	.0189	+1.09	+.17	+1.76	+1.11	107.5	106.5	212.9	129.6
21	3.6	4.2	4.5	3.3	3.4	47	35	.0216	.0241	.0223	.0230	+.20	−.54	−.02	−.21	90.6	102.7	161.3	116.4
22	2.3	3.1	3.0	2.5	3.1	56	43	.0191	.0229	.0174	.0214	+1.05	−.19	+1.70	+.26	101.3	106.0	204.1	125.8
23	5.5	6.5	(4)	4.7	4.3	33	26	.0263	.0271	.0292	.0277	−1.08	−1.26	−1.74	−1 42	77.4	90.0	13.2	−6.4
24	3.2	4.1	4.0	3.0	3.2	43	-------	.0216	.0228	.0207	.0226	+.19	−.16	+.47	−.12	90.6	98.4	152.8	94.9
25	4.1	4.7	(4)	3.5	3.3	41	33	.0228	.0224	.0231	.0228	−.16	−.07	−.26	−.16	87.2	99.8	146.7	102.5
26	3.5	4.0	5.0	3.5	3.4	42	36	.0226	.0231	.0227	.0231	−.10	−.24	−.15	−.26	87.4	94.8	85.9	74.8
27	4.1	5.2	(4)	3.4	3.9	41	29	.0223	.0241	.0236	.0256	−.03	−.53	−.40	−.92	87.4	97.9	80.0	82.8
28	4.8	6.1	4.8	3.4	4.1	36	26	.0234	.0245	.0228	.0272	−.35	−.65	−.16	−1.30	86.8	96.4	153.4	51.5
29	-------	6.1	-------	-------	3.9	-------	27	-------	.0258	-------	.0266	-------	−.98	-------	−1.16	-------	95.5	-------	9.2
30	3.2	4.8	3.9	3.0	3.6	44	37	.0226	.0241	.0203	.0245	−.10	−.53	+.62	−.63	89.6	100.4	224.2	85.1
31	3.1	3.3	3.5	2.8	2.9	49	47	.0202	.0214	.0195	.0205	+.64	+.26	+.89	+.54	106.6	109.1	197.1	113.5
32	3.8	4.7	4.5	3.3	3.2	45	33	.0240	.0221	.0227	.0225	−.51	+.05	−.15	−.08	84.6	98.8	115.6	93.5
33	3.5	5.0	4.8	3.3	3.0	46	34	.0248	.0234	.0227	.0219	−.71	−.34	−.15	+.10	75.2	92.9	30.8	11.1
34	3.1	3.8	4.0	3.1	3.3	47	40	.0221	.0236	.0215	.0227	+.03	−.40	+.22	−.15	91.4	98.9	84.7	35.7
35	3.4	3.8	(4)	3.4	3.2	47	42	.0247	.0234	.0229	.0224	−.68	−.34	−.20	−.05	85.8	102.3	87.2	64.3
36	3.4	3.6	4.5	3.2	2.7	45	40	.0240	.0212	.0215	.0192	−.51	+.31	+.22	+.99	88.1	100.1	109.5	73.5
37	2.7	3.0	3.6	2.8	2.9	56	49	.0214	.0222	.0194	.0200	+.26	-------	+.93	+.71	96.8	108.2	164.8	95.0
38	2.8	2.2	3.3	2.7	2.7	60	53	.0212	.0210	.0198	.0188	+.33	+.38	+.79	+1.15	97.6	113.2	148.0	94.9
39	5.9	5.7	5.7	3.8	4.1	30	27	.0247	.0247	.0248	.0277	−.68	−.68	−.71	−1.42	80.8	94.2	76.1	39.6
40	3.8	4.8	(4)	3.6	4.0	42	41	.0262	.0268	.0240	.0261	−1.07	−1.20	−.52	−1.04	80.3	91.4	19.0	−9.6

[1] Penetrations at various temperatures as follows:
32° F., 200 gm., 60 sec.
39.2° F., 200 gm., 60 sec.
77° F., 100 gm., 5 sec.
100° F., 100 gm., 5 sec.
115° F., 50 gm., 5 sec.

[2] All 85-100 penetration asphalts too soft for penetration at 115° F. in standard container.

[3] Penetration-temperature susceptibility factor.

[4] Material too soft for penetration at 115° F.

$$\text{Susceptibility factor} = \frac{\text{penetration } 115^\circ \text{ F., 50 gm., 5 sec.} - \text{penetration } 32^\circ \text{ F., 200 gm., 60 sec.}}{\text{penetration } 77^\circ \text{ F., 100 gm., 5 sec.}}$$

and required that it should not be more than 4.2 for asphalts of 40-70 penetration. The use of this factor for asphalts of normal construction grade is limited by the fact that the material may be too soft for a penetration test at the temperature of 115° F. As indicated in table 11, all of the 85-100 penetration asphalts were too soft for a penetration test at this temperature, while of the 50–60 asphalts, only 27 of the 39 materials could be tested. Of these 27 samples, there were 15 that would not meet the requirement for a maximum susceptibility factor of 4.2. The other 50–60 penetration materials, too soft for penetration at 115° F., would also fail, so that a total of only 12 out of the 39 asphalts tested would meet the requirement.

Two other susceptibility factors have been suggested that are based on the following relationships:

$$\text{a.—S. F.}=\frac{\text{penetration } 77^\circ \text{ F., } 100 \text{ gm., } 5 \text{ sec.}}{\text{penetration } 32^\circ \text{ F., } 200 \text{ gm., } 60 \text{ sec.}}$$

$$\text{b.—S. F.}=\frac{\text{penetration } 100^\circ \text{ F., } 100 \text{ gm., } 5 \text{ sec.}}{\text{penetration } 77^\circ \text{ F., } 100 \text{ gm., } 5 \text{ sec.}}$$

Limits for these factors have not been proposed for specification requirements, and the values in table 11 are shown for comparative purposes only.

However, one State has recently adopted a susceptibility factor based on the ratio of the penetration at 39.2° F., 200 grams, 60 seconds to the penetration at 77° F., 100 grams, 5 seconds. The requirement states that the penetration at 39.2° F. must be greater than 30 percent of the penetration at 77° F. For the 50–60 grade, this limit would cause the rejection of four out of five California asphalts. For the 85–100 grade, four out of five California asphalts together with four asphalts from Oklahoma and one from Texas would also fail to meet this requirement.

Penetration-temperature relationship.—The results of the penetration tests made under a load of 100 grams for 5 seconds at temperature intervals of 9° F. (5° C.) from a minimum temperature of 32° F. to a maximum of 104° F. for the 50–60 asphalts and to a maximum of 95° F. for the 85–100 asphalts, are shown in table 10. These tests were made in the standard 3-ounce container and the depth did not permit higher test temperatures to be used. Some materials of the 50–60 grade were too soft to obtain a penetration even at the temperature of 104° F.

It was found that when the values are plotted to scale there is a straight-line relation between the temperature of test and the logarithm of the corresponding penetration. A few typical examples of this relation are shown in figure 4.

Penetration index.—J. Ph. Pfeiffer and P. M. Van Doormal [2] [3] have described a method of classifying asphalts by means of a "penetration index." In calculating this penetration index, the assumption is made that the penetration at the softening point is 800 for all asphalts. This value is used in an equation for calculating the penetration-temperature susceptibility factor as follows:

$$P\,T\,S=\frac{\log 800-\log p}{t-77}$$

where $P\ T\ S$=penetration-temperature susceptibility factor

p=penetration at 77° F., 100 gm., 5 sec.
t=softening point, °F.

This equation is the slope of the log-penetration-temperature line for the assumed penetration value of 800 and penetration, p, at the temperatures t and 77° F. respectively. The $P\,T\,S$ value is then used in calculating the penetration index by the equation

$$PI=\frac{30}{1+90(P\,T\,S)}-10$$

Values of the penetration-temperature susceptibility factor ($P\,T\,S$) and penetration index calculated in this manner are given in table 11.

For a given penetration at 77° F. the softening point is the only variable in the equation for the value of the $P\,T\,S$, as determined by Pfeiffer and Van Doormal, and thus, for asphalts of the same penetration, the value of the penetration index is dependent only on the softening point.

Since the log-penetration-temperature curves of the 50–60 and 85–100 penetration asphalts had been made, the penetration at the temperature of the softening point of each material could be found by extending the straight line up to a temperature equal to the softening point, as shown in figure 4. It was found that these values ranged from approximately 540 to 1,060 for the 50–60 grade and from approximately 620 to 1,300 for the 85–100 grade. The calculated values for the penetration at the temperature of the softening point were partially checked for the 85–100 penetration asphalts by determining the penetration of samples 14, 19, and 39, at 115° F. In determining these values, a special needle 6 inches long but otherwise conforming to the specification of the standard needle was used. Large samples of the selected asphalts were carefully prepared and held in the water bath at 115° F. until the thermometer immersed at the center of the sample came to test temperature The values obtained are shown in figure 5.

Since the log-penetration-temperature curves of the 50–60 and 85–100 penetration asphalts assume the form of straight lines, a true indication of their susceptibility to change of temperature is found in the slope of the lines. The slope can be calculated as follows:

$$\text{Slope}=M=\frac{\log.p_2-\log p_1}{t_2-t_1}$$

where p_2 and p_1 are penetrations at the two temperatures t_2 and t_1, respectively. The slope can be calculated readily from the penetration-temperature curves by finding the temperatures of the asphalt at penetrations of 10 and 100. Then

$$\text{Slope}=\frac{\log 100-\log 10}{t_2-t_1}=\frac{2-1}{t_2-t_1}=\frac{1}{t_2-t_1}$$

Where t_2=temperature, °F., corresponding to a penetration of 100
t_1=temperature, °F., corresponding to a penetration of 10.

The values of the slope for the 50–60 and 85–100 penetration asphalts are given in table 11.

MAJORITY OF SAMPLES FAILED TO MEET FLUIDITY FACTOR REQUIREMENTS

With these values of slope, which are also the true values of the penetration-temperature susceptibility factor, the exact values of the penetration index can be calculated. In general, the exact penetration index values, also given in table 11, show a wider range than those calculated on the basis of an assumed penetration

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[2] Journal Institute Petroleum Technologists 1936, 22, 414.
[3] Classifying Asphalts by Means of Penetration Index. Reprint from National Petroleum News. Refining Technology. February 1938.

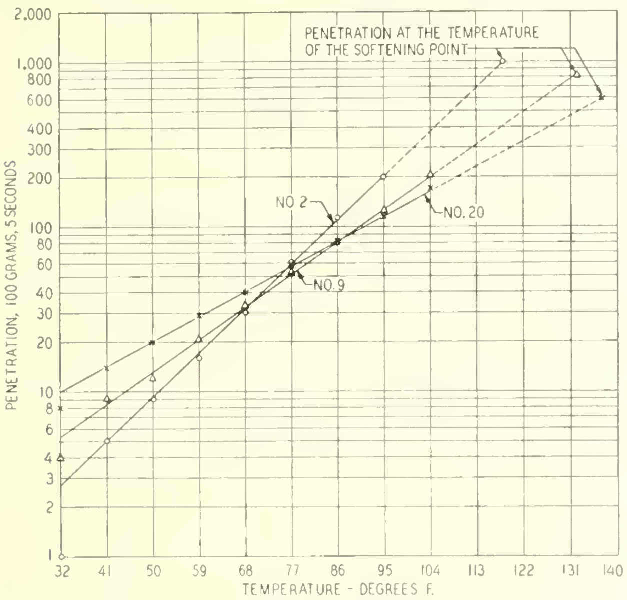

Figure 4.—Penetration-Temperature Curves for 50–60 Penetration Asphalts.

of 800 at the softening point temperature. Although the penetration indexes using the true values for the penetration-temperature susceptibility factor provide sharper differentiation among the asphalts used in this report, it is believed that the penetration-temperature susceptibility as determined by the slope of the log-penetration-temperature curve, is a true measure of susceptibility. The penetration index, whether based on the assumption of Pfeiffer and Van Doormal or on the true value of the penetration-temperature susceptibility factor, is an empirical equation that distorts the results and is not a true measure of susceptibility.

Fluidity factor: According to Joseph Zapata,[4] the fluidity factor was first proposed as a means for identifying the source of the crude from which the asphaltic material was refined. Early investigators found that asphalt cements from various sources were represented by typical viscosity temperature curves and that the maximum deviation in viscosity for these curves occurred at 135° C. (275° F.). With this as a basis the penetration at 77° F. and the factor $\frac{P}{100}$ were introduced to give an empirical formula:

$$\text{Fluidity factor} = (V-P)\times\frac{P}{100}$$

where V=Furol viscosity at 135° C. (275° F.)
P=penetration at 25° C. (77° F.).

Tests limits for fluidity factor when inserted in specifications may be considered, therefore, a means of controlling the source of an asphalt by its susceptibility to change in temperature. While the Furol viscosity of the asphalt at high temperatures may be of value for governing the temperatures of the mixing operation, a susceptibility factor based on the viscosity at this high temperature might not give an adequate indication of the change in consistency of the material over the temperature range of the pavement in service. Furthermore, as indicated by Zapata[5] in 1937, the initial purpose of the test can be voided by blending and blowing oils to meet the established limits for fluidity factors.

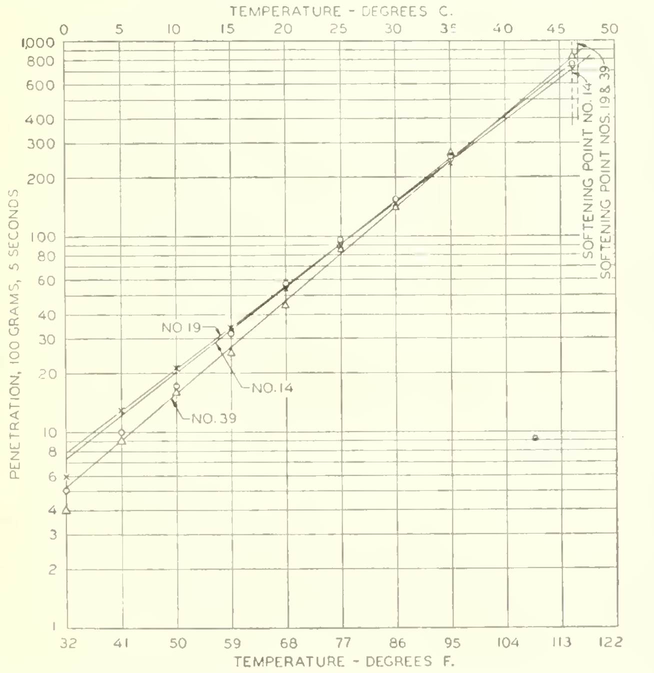

Figure 5.—Penetration-Temperature Curves Showing the Actual Penetration at the Temperature of the Softening Point; 85–100 Penetration Asphalts.

Furol viscosities of all the 50–60 and 85–100 asphalts at 275° F. were determined and the fluidity factors calculated. The Furol viscosities are given in table 10 and the fluidity factors in table 11.

A minimum limit of 140 for the fluidity factor has been included in certain specifications for asphalt cements having penetrations between 40 and 100. Of the 50–60 penetration grade, 22 asphalts out of 39 failed to meet this requirement. Those having fluidity factors below 140 include all of the California asphalts, one Mexican asphalt, the Colombian asphalt, 5 out of 6 Venezuelan asphalts, 3 of the 6 Oklahoma asphalts, and the Kansas, Texas, and Kentucky asphalts. Those asphalts having fluidity factors greater than 140 are the other Mexican, the Mexican and domestic blends, Arkansas, and a few other Mid-continent asphalts. There were 34 out of 40 samples of the 85–100 penetration grade that did not meet this minimum requirement of 140. Those meeting the requirement were 5 of the 6 Mexican asphalts and 1 of the 6 Venezuelan asphalts.

It is apparent, because of the large number of asphalts of the 85–100 grade that were lower than the specified minimum requirement, that the fluidity factor is materially affected by the consistency of the material even within the penetration limits of 40 to 100. In order to determine the effect of penetration on the fluidity factor, a set of Mexican asphalts received from producer 10 and a set of Texas asphalts received from producer 27 were tested for penetration at 77° F. and Furol viscosity at 275° F. and the fluidity factors were determined. These values are shown graphically in figure 6. The fluidity factors for the Mexican asphalts were fairly constant between the 40–100 penetration range and then the values decreased rapidly. Although the fluidity factors of all the Texas asphalts were below 140, there is a relatively wide range in these values between the penetration limits of 40 to 100.

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[4] The Fluidity Factor Test, Proceedings Association of Asphalt Paving Technologists. January 30, 1935.

[5] A Study of Bituminous Material Weathering Tests. Proceedings Association of Asphalt Paving Technologists, January 1937.

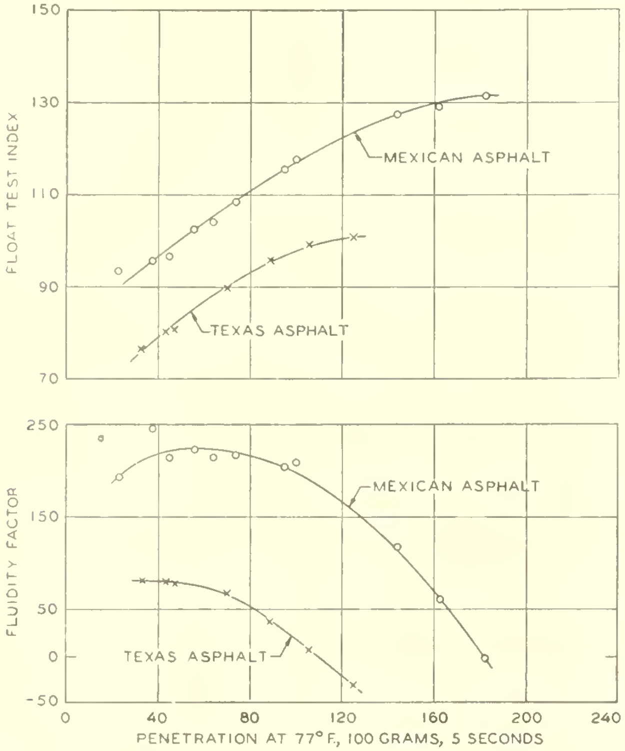

Figure 6.—Relations Between Penetration and Fluidity-Factor and Between Penetration and Float-Test Index of Asphalts of Different Penetrations From the Same Sources.

FLUIDITY FACTOR AND FLOAT TEST REQUIREMENTS HIGHLY RESTRICTIVE

Float test index.—The values for the float test, shown in table 10, were used to determine the float test index of the asphalts of both grades and the results are shown in table 11. The float test index is determined by the following formula:

$$\text{Float test index} = \sqrt{F \times P}$$

where F=Float test at 176° F. (80° C.) in seconds.

P=Penetration at 77° F. (25° C.) 100 grams, 5 seconds.

This index is used, like the fluidity factor, in an attempt to measure the susceptibility of an asphalt to change in temperature. Certain specifications have required that the float test index shall not be less than 90. This requirement applies to asphalts of 40–70 penetration but the indexes for the asphalts of both grades used in this study have been calculated for comparative purposes.

As shown in table 11 there were 24 asphalts of the 50–60 grade that did not meet the float test index requirement of 90 plus. All but 8 of the 24 asphalts were the same as those failing to meet a minimum fluidity factor of 140. Five of the twenty-four failing to pass the float test index requirement, however, did pass the fluidity factor requirement. In contrast to the fluidity factor, which is more restrictive for the softer grades of asphalts, a float test index of 90 is more favorable to the asphalts of higher penetration, since only one 85–100 penetration asphalt failed to meet the requirement. This, however, might be expected since the limit of 90 was specified for asphalts of 40–70 penetration only. This is further indicated by the float test index values for the same series of Mexican and Texas asphalts referred to in the discussion of the fluidity factor. These values are shown also in figure 6. For each series of asphalts, the difference in float test index for materials of 40 penetration and materials of 100 penetration was approximately 20. Although all the Mexican asphalts had float test indexes greater than 90, the grades of Texas asphalts harder than 70 penetration failed to meet this requirement.

The various susceptibility factors, including the penetration index as determined by Pfeiffer and Van Doormal, as given in table 11, except the one involving penetration at 115° F., have been plotted in figures 7 and 8. In these figures, the slope of the log-penetration-temperature curve, which is a true index of susceptibility, has been plotted against the other susceptibility factors. It is apparent from these charts that none of the empirical factors is more than an approximate measure of susceptibility.

While these various susceptibility factors might be used to advantage in controlling the uniformity of supply for particular jobs, a study of table 11 will indicate that they can be used for the identification of sources of the base petroleums. In many cases by special processing, which may be injurious to some asphalts, materials could undoubtedly be produced to meet any of the requirements that have been proposed for these different factors. However, since materials representing the most susceptible asphalts in this investigation have had satisfactory service behavior under climatic conditions as adverse as those to which the less susceptible materials have been exposed, it is felt that a designated limit for a susceptibility factor that restricts the use of such materials is not a rational requirement.

With the few exceptions already mentioned, all of the asphalts had much higher ductilities than were required by the standard specifications. However, the ductility test, as made under normal testing conditions, furnishes little comparative information on the ductile characteristics of different asphalts.

In order to make a more comprehensive study of the ductility test, a special ductility machine was designed and built. The bath has approximately twice the length of the ordinary bath, and the temperature of the bath at both high and low temperatures can be controlled adequately. A wide range in the speed of pull has been provided. In this machine five tests may be made at the same time and ductilities up to 250 centimeters may be measured. The design is such that there is little shock in starting and a uniform speed may be maintained during the test.

DUCTILITIES OF ASPHALTS DETERMINED FOR SEVERAL TEMPERATURES

In making the ductility tests in this investigation, A. S. T. M. Method D113–35 was followed, and the precautions as to time of cooling, and the minimum and maximum times at test temperatures were carefully observed. At test temperatures of 59° F. and higher, it was necessary to control the gravity of the bath within 0.005 of the specific gravity of the asphalt in order to keep the thread of asphalt in a horizontal plane. The lower test temperatures were maintained by ice, and at 32° F. both ice and salt were used. The large capacity of the bath made it possible to control its temperature within ±0.9° F. at the test temperatures of 32°, 39.2°, and 41° F. without disturbing the thread of asphalt by any movement of the ice.

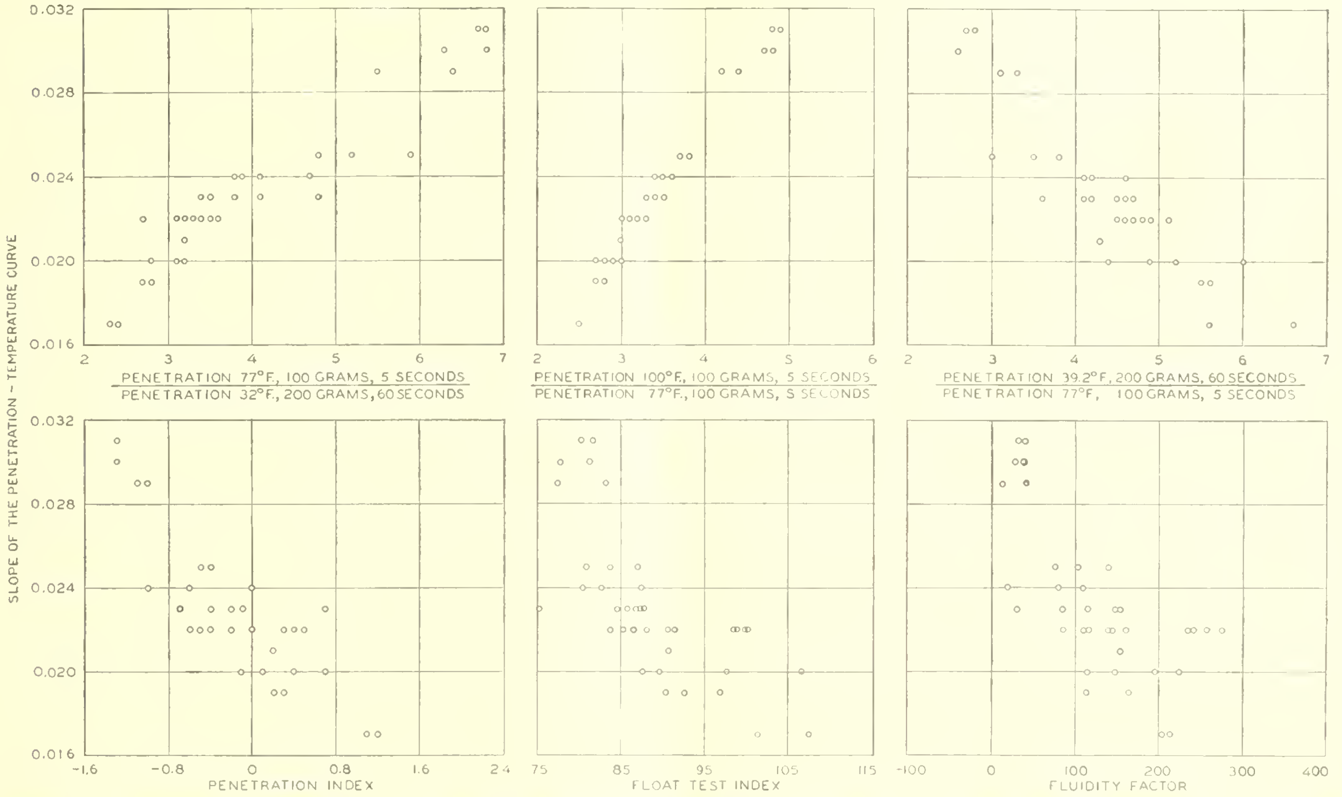

FIGURE 7.—COMPARISON OF THE SLOPE OF THE PENETRATION-TEMPERATURE CURVES TO VARIOUS OTHER SUSCEPTIBILITY FACTORS FOR THE 50–60 PENETRATION ASPHALTS.

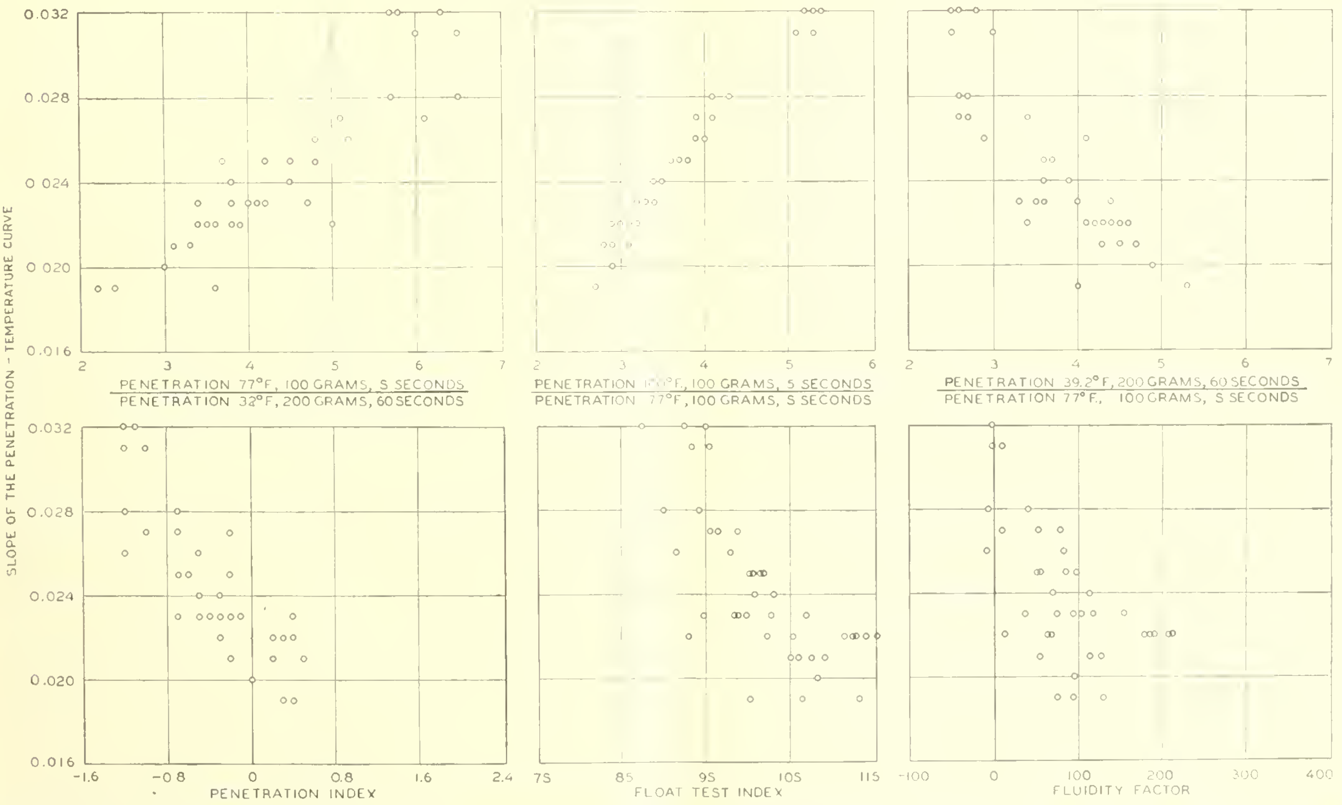

FIGURE 8.—COMPARISON OF THE SLOPE OF THE PENETRATION-TEMPERATURE CURVES TO VARIOUS OTHER SUSCEPTIBILITY FACTORS FOR THE 85–100 PENETRATION ASPHALTS.

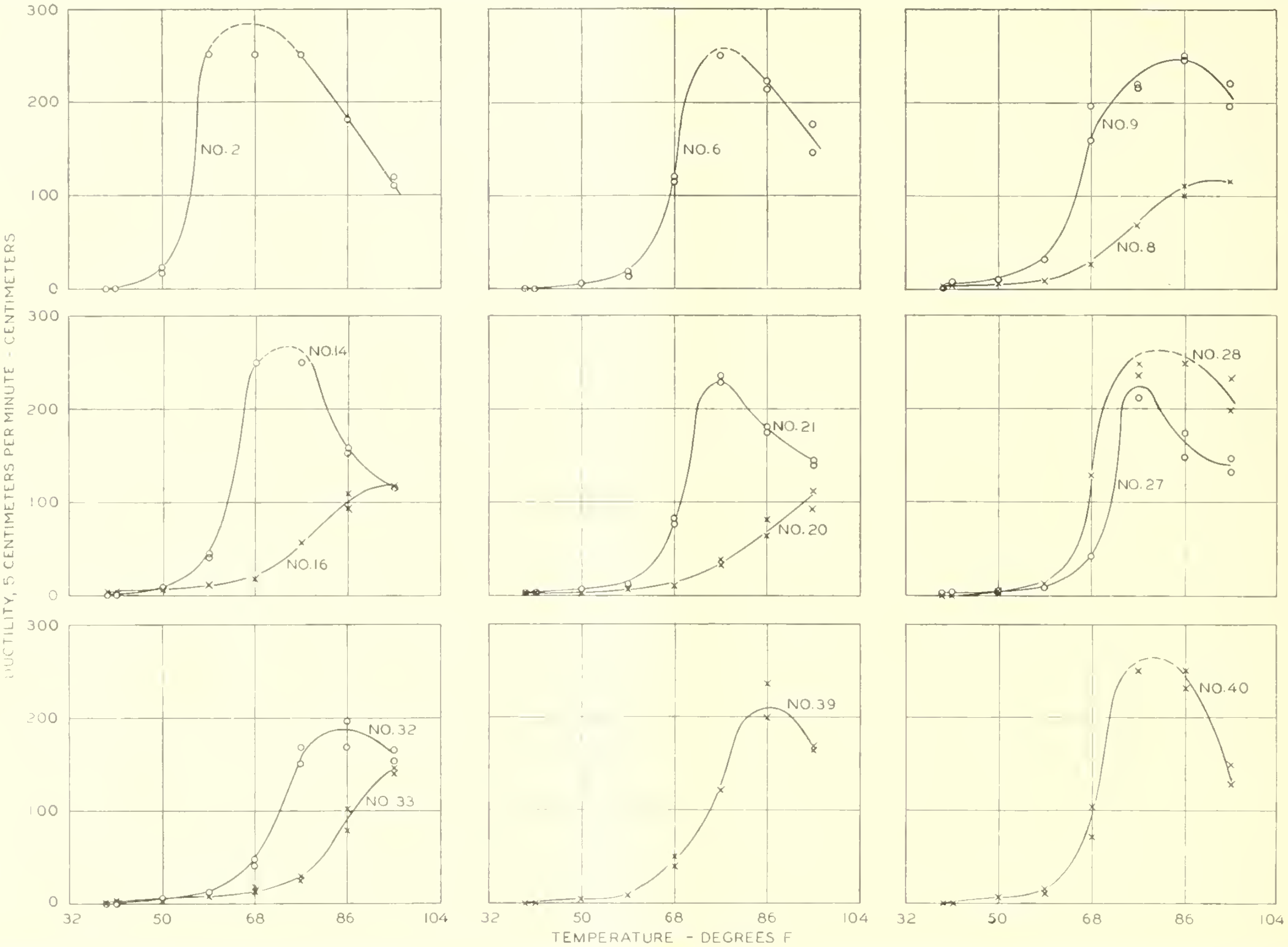

Figure 9.—Relation Between Ductility, at 5 Centimeters per Minute, and Test Temperature of Selected Samples of 50–60 Penetration Asphalts.

From the time that the ductility test was first suggested as a control test for asphalt cements, arguments have been advanced for and against the adequacy and value of the test. Even after requirements for ductility had been generally adopted for specification purposes, the differences of opinion continued and, although a test method was first proposed by the A. S. T. M. in 1921, it was not finally adopted as a standard until 1935.

For many years, the test has been made at 77° F. with a rate of pull of 5 centimeters per minute, although in recent years other temperatures have been used. In this study all the asphalts were tested at a standard temperature of 77° F., and at the low temperatures which have been most frequently proposed, 32° and 39.2° F. At 39.2° F. three rates of pull were used and two rates of pull at 32° F. The results are shown in table 12.

In order to determine the effect of temperature and rate of pull on ductility, 14 materials of the 50–60 grade were selected for special study. These asphalts, where possible, were materials of high and low ductility at 77° F. from each of the sources. The values for the ductility of these selected asphalts are given in table 13, and the data are shown graphically in figures 9 and 10.

Figure 9 shows the effect of temperature on the ductility when the rate of pull is 5 centimeters a minute. Figure 10 shows the same data except that the penetration of the asphalt at the test temperature has been plotted against the ductility of the asphalt at the same temperature.

Figure 9 shows that there is a definite temperature at which all the asphalts had a maximum ductility. Although this is indeterminable for several of the materials because their ductility exceeded 250 centimeters, in general it ranges from approximately 68° F. for sample 2 to over 95° F. for samples 16, 20, and 33. Sample 2 showed the most rapid change in ductility with increase in temperature. This material had no measurable ductility at 41° F., 19 centimeters at 50° F., and 250 centimeters plus at 59° F. At 77° F., it still had a ductility of 250 centimeters plus. The other asphalts, except samples 8, 16, 20, and 33, had a relatively low ductility up to 59° F. and then the ductility increased rapidly between 59° F. and 77° F. Asphalts 8, 16, 20, and 33 had a more gradual increase in ductility with rise in test temperature up to 95° F. With the exception of samples 8, 16, 20, and 33, the curves indicate that at temperatures higher than 95° F. the ductilities of all the asphalts decreased rapidly.

As shown in figure 10, the susceptibility of the individual asphalts to changes in temperature tended to alter somewhat the shape of the curves. It will be seen that sample 2 retained a very high ductility at a penetration much lower than any of the other asphalts. From a penetration of 16 to a penetration of 200 this material had a ductility of more than 100 centimeters. With a decrease in penetration below 16, however, this

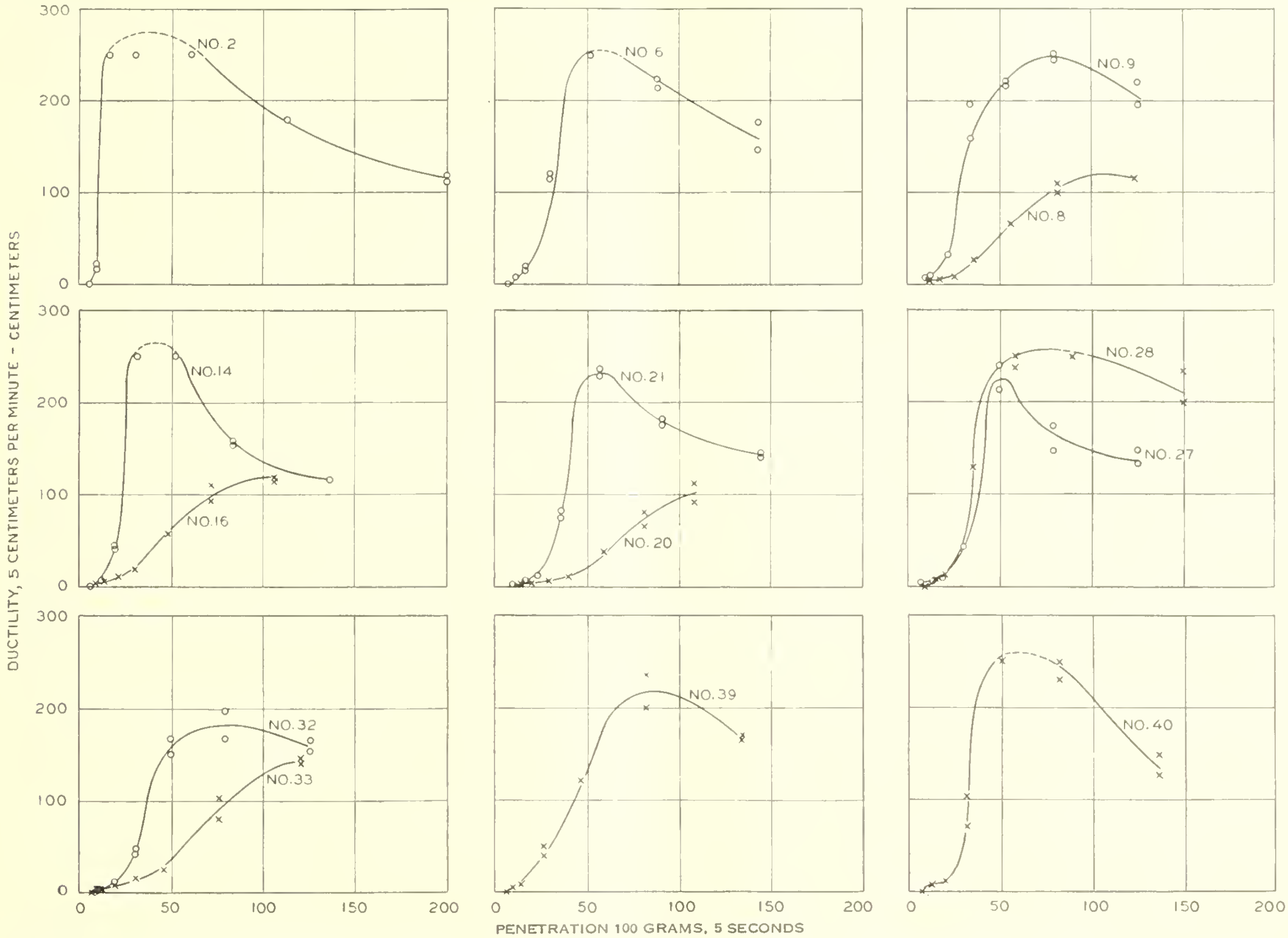

Figure 10.—Relation Between Ductility, at 5 Centimeters per Minute, and Penetration, 100 Grams, 5 Seconds, of Selected Samples of 50–60 Penetration Asphalts, Tested at Various Temperatures.

material showed an abrupt loss in ductility. These curves indicate that while asphalts may have identical ductilities when tested at one consistency, they may differ widely when tested at another consistency. The curves show that both the source and the processing have a pronounced influence on the ductility characteristics of the asphalt.

DUCTILITIES OF ASPHALTS AT LOW TEMPERATURES GREATLY INFLUENCED BY RATE OF PULL

The data shown in table 14 and plotted in figure 11 indicate that the ductility test can be used to identify the source of the base material and the uniformity of the refining process. An entire set of Mexican asphalts, of which the 50–60 and 85–100 penetration asphalts identified as sample 10 were a part, was tested for penetration and ductility at various test temperatures. The penetrations of the asphalts at the temperature of the ductility tests were measured at 59°, 77°, and 95° F., and values at 50°, 68°, and 86° F. were taken from the temperature-penetration curves. The slope of these penetration-temperature curves was the same for all the grades. The data and the curve show that the ductilities of the asphalts in this set were essentially the same when the penetrations of the materials at their respective test temperatures were the same. This figure further shows that ductility is dependent upon the consistency of the material at the test temperature.

In order to determine the effect of the rate of pull on the ductility of asphalts, a study was made of the fourteen 50–60 penetration asphalts previously mentioned. Preliminary tests indicated that a test temperature somewhat lower than 77° F. would provide a much better comparison, and the temperature of 59° F. accordingly was selected. The rates of pull were 10, 7.5, 5, 2.5, and 1 centimeters per minute. The data are given in table 13 and the results obtained on the individual materials are plotted in figure 12. Sample 2 had ductilities greater than 250 centimeters for all rates of pull. Asphalts 8, 16, 20, and 33, all of which had ductilities under 70 centimeters in the standard test at 77° F. showed relatively small increases in ductility as the rate of pull was decreased. All of the other samples had appreciably higher ductilities at rates of pull of 2.5 and 1 centimeters. Sample 14 showed the greatest increase in ductility at 5 centimeters over that at 10 centimeters, and at a rate of 2.5 centimeters this material had the highest ductility of all samples except sample 2.

The tests made on these asphalts, to show the effect of temperature and rate of pull on their ductility, indicate that there were inherent differences in these materials that were not made evident by the routine tests for ductility. Whether the ductility tests made under various temperature conditions and at different rates of pull are of any practical significance it is impos-

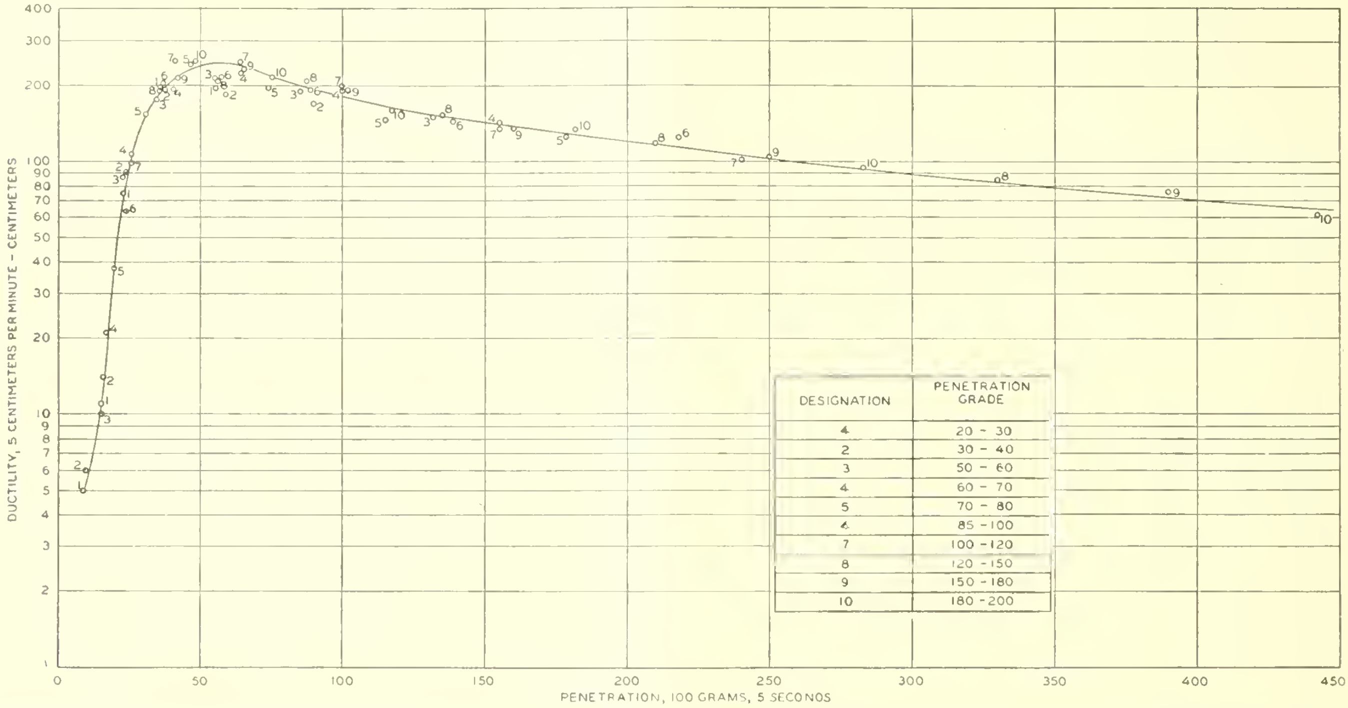

FIGURE 11.—RELATION BETWEEN PENETRATION AND DUCTILITY OF VARIOUS GRADES OF MEXICAN ASPHALT TESTED AT VARIOUS TEMPERATURES.

TABLE 12.—*Ductility and toughness tests on the 50–60 and 85–100 penetration asphalts*

Identification No.	Ductility												Toughness test at 32° F.	
	50–60 grade						85–100 grade						50–60 grade	85–100 grade
	At 77° F.	At 39.2° F.			At 32° F.		At 77° F.	At 39.2° F.			At 32° F.			
	5 cm. per minute	5 cm. per minute	1 cm. per minute	¼ cm. per minute	1 cm. per minute	¼ cm. per minute	5 cm. per minute	5 cm. per minute	1 cm. per minute	¼ cm. per minute	1 cm. per minute	¼ cm. per minute		
	Cm.	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*
1	250+	[1] 0	8.3	160+	[1] 0	[1] 0.3	223	[1] 0	160.0	250+	[1] 0.3	250+	5.0	10.0
2	250+	[1] 0	7.5	160+	[1] 0	[1] .3	193	[1] 0	188.0	250+	[1] .3	250+	5.0	12.5
3	250+	[1] 0	8.5	160+	[1] 0	[1] 0	204	[1] 0	170.0	250+	[1] .8	250+	5.0	12.5
4	250+	[1] 0	9.5	160+	[1] 0	[1] 0	227	[1] 0	130.0	250+	[1] .3	220.0	7.5	12.5
5	250+	[1] 0	[1] 0	26.0	[1] 0	[1] 0	197	[1] 0	15.5	215.0	[1] .3	8.5	7.5	12.5
6	250+	[1] 0	4.8	8.5	[2] .5	4.4	185	5.5	13.3	32.0	5.0	10.8	10.0	7.5
7	197	[1] 1.0	7.9	27.5	4.9	7.1	220	9.7	47.5	100.0	9.5	24.5	15.0	22.5
8	68	3.5	4.4	6.5	2.8	3.3	102	4.5	7.8	10.0	4.5	6.0	15.0	35.0
9	218	[1] 1.0	6.5	14.5	4.0	6.5	230	9.0	35.0	123.0	8.5	32.0	12.5	20.0
10	215	[1] 1.0	6.9	18.8	4.5	6.5	192	7.2	37.0	79.0	9.0	22.8	12.5	12.5
11	180	[1] .5	5.9	10.5	3.5	4.8	209	9.0	20.8	76.0	7.5	17.0	10.0	20.0
12	250+	[1] .5	6.3	16.0	3.0	6.8	242	8.0	56.0	102.0	9.5	36.0	10.0	12.5
13	140	3.7	4.9	6.5	3.3	4.8	192	6.5	15.8	22.0	7.0	13.8	20.0	20.0
14	250+	[1] 0	5.5	11.5	[2] .3	4.8	192	5.2	20.3	87.0	6.0	14.0	10.0	10.0
15	181	[2] 1.0	5.6	10.5	3.8	5.0	187	6.0	19.5	95.0	6.3	18.0	12.5	10.0
16	57	3.7	4.5	6.8	[2] 2.5	4.3	107	7.0	14.8	23.0	6.8	13.3	12.5	15.0
17	250+	[1] .5	5.9	10.0	[2] 1.0	5.5	191	6.0	29.3	119.0	7.3	22.0	10.0	12.5
18	250+	4.0	5.5	10.0	3.9	5.3	196	6.0	15.3	47.0	6.5	9.8	10.0	12.5
19	220	[1] 0	4.8	8.8	[2] .5	[1] .3	179	5.2	11.0	33.5	5.0	8.5	10.0	7.5
20	36	3.6	4.0	5.0	3.1	3.9	139	5.0	7.0	10.0	5.0	6.0	22.5	17.5
21	232	3.8	4.9	7.5	4.0	4.5	178	5.2	12.8	20.5	5.3	10.8	7.5	12.5
22	96	3.7	4.8	6.8	3.8	4.5	211	5.5	13.3	26.8	6.3	12.5	20.0	15.0
23	202	[1] 0	3.1	5.0	[2] .5	[2] .5	223	[1] .8	7.8	12.8	4.0	6.3	10.0	10.0
24	84	2.8	4.1	5.3	[2] 2.0	3.4	162		5.3	7.8	3.8	4.5	7.5	10.0
25	116	3.3	5.3	8.5	3.5	4.3	152	5.3	10.5	18.5	5.0	8.3	7.5	7.5
26	78	3.2	4.3	6.3	3.3	3.9	172	4.6	10.0	12.8	4.5	7.8	12.5	10.0
27	226	3.8	4.5	6.5	[2] .8	4.0	164	[2] 1.0	12.8	29.0	4.8	9.5	12.5	10.0
28	244	[1] 0	4.8	7.5	[2] .8	[2] 2.9	184	[2] 1.0	18.0	25.5	5.0	11.3	10.0	5.0
29							200	[2] .2	9.5	17.3	4.0	6.3		5.0
30	170	[1] 0	4.3	6.3	3.0	3.8	163	5.0	8.3	15.5	4.8	7.0	15.0	10.0
31	41	3.2	4.0	5.8	[1] .3	4.5	101	5.4	7.5	10.3	4.8	7.0	20.0	15.0
32	168	[2] .8	4.3	6.3	[2] 1.3	3.4	173	4.0	7.8	11.8	4.0	6.5	10.0	10.0
33	27	[1] .8	2.6	4.8	[2] .3	[2] 1.0	125	3.5	5.8	7.5	3.5	5.3	10.0	10.0
34	112	[1] 0	4.5	7.0	3.4	4.0	170	5.0	11.3	20.8	5.5	7.8	20.0	10.0
35	219	4.1	5.3	8.8	3.8	4.6	186	4.5	10.8	19.8	6.0	9.8	15.0	12.5
36	190	3.7	5.3	7.8	3.8	4.5	115	4.3	6.5	7.0	4.5	5.0	15.0	10.0
37	137	3.7	5.0	7.3	3.3	4.3	193	5.8	9.8	16.0	5.5	9.0	10.0	20.0
38	120	4.0	5.1	7.3	4.0	5.1	120	5.7	9.0	11.3	5.5	8.3	15.0	22.5
39	121	[1] 0	[2] 1.5	[2] 2.0	[1] 0	[1] 0	141	[1] 0	7.0	13.5	[2] .3	5.8	5.0	10.0
40	250+	[1] 0	4.5	9.5	[1] .5	[1] 2.0	250+	[1] 0	14.0	28.0	6.5	15.5	10.0	15.0

[1] Test specimen broke at shoulder of mold.

[2] Test specimen broke at smallest cross section.

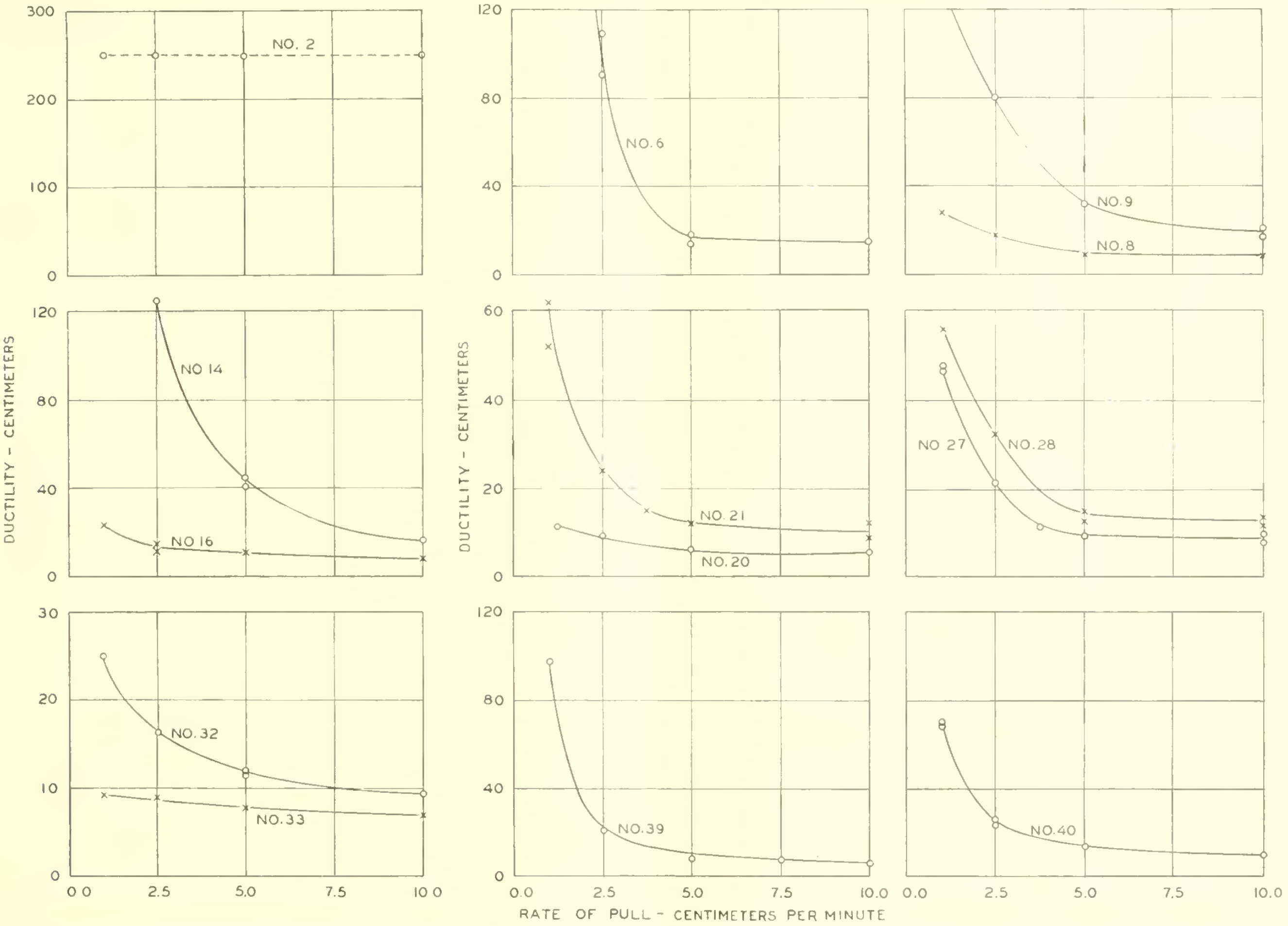

FIGURE 12.—RELATION BETWEEN DUCTILITY AND RATE OF PULL OF SELECTED SAMPLES OF 50–60 PENETRATION ASPHALTS TESTED AT 59° F. (15° C.)

TABLE 13.—*The effect of varying the temperature of test and the rate of pull on the ductility of selected samples of 50–60 penetration asphalt*

Sample No.	Ductility, 5 cm. per minute [1]								Ductility at 59° F.[1]			
	At 39.2° F.	At 41° F.	At 50° F.	At 59° F.	At 68° F.	At 77° F.	At 86° F.	At 95° F.	10 cm. per minute	5 cm. per minute	2.5 cm. per minute	1 cm. per minute
	Cm.	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*	*Cm.*
2	0	0	19.0	250+	250+	250+	180	114	250+	250+	250+	250+
6	0	0	6.4	16.0	118	250+	219	161	15.0	16.0	100.0	250+
8	3.5	4.3	5.8	9.3	27	68	106	115	8.5	9.3	18.0	28.0
9	0	5.3	7.8	32.0	178	218	248	208	19.0	32.0	80.0	136.0
14	0	0	7.5	43.0	250+	250+	155	115	17.3	43.0	125.0	200+
16	3.7	4.5	6.5	11.0	20	57	102	117	8.5	11.0	14.0	23.3
20	3.6	4.0	4.5	6.5	11	36	73	102	5.8	6.5	9.5	11.5
21	3.8	4.0	6.0	12.3	79	232	178	143	10.8	12.3	24.0	57.0
27	3.8	3.8	5.7	9.7	43	226	162	141	9.5	9.7	21.5	48.0
28	0	0	6.3	14.0	130	244	250+	217	13.4	14.0	32.0	56.0
32	0	0	5.0	11.8	45	159	183	159	9.5	11.8	16.5	25.0
33	0	3.0	4.0	7.8	16	27	90	143	7.0	7.8	9.0	9.3
39	0	0	4.0	8.3	45	121	218	168	7.0	8.3	21.5	98.0
40	0	0	6.0	15.0	88	250+	240	138	10.5	15.0	25.5	70.0

[1] Results are averages of 2 or more tests.

TABLE 14.—*The ductility-penetration relationships of various grades of Mexican asphalt under variable temperature test conditions*

Penetration grade	Temperature of test											
	50° F.		59° F.		68° F.		77° F.		86° F.		95° F.	
	Penetration [1]	Ductility	Penetration	Ductility	Penetration [1]	Ductility	Penetration	Ductility	Penetration [1]	Ductility	Penetration	Ductility
		Cm.		*Cm.*		*Cm.*		*Cm.*		*Cm.*		*Cm.*
20–30	6	0	9	5	15	11	23	75	36	195	55	192
30–40	10	6	16	14	24	91	38	194	59	185	90	170
50–60	15	10	23	87	35	178	55	215	85	190	132	150
60–70	17	21	26	107	41	194	64	225	100	193	155	142
70–80	20	38	31	154	47	245	74	197	115	147	179	126
85–100	24	64	37	205	57	218	89	192	139	145	218	126
100–120	26	99	41	250+	64	250+	100	197	155	134	240	101
120–150	36	193	56	210	87	210	135	153	210	119	330	85
150–180	42	215	65	233	102	192	160	135	250	104	390	76
180–200	48	250+	75	217	117	160	182	134	283	95	442	62

[1] Penetrations taken from log-penetration-temperature curves.

sible to say, but the data and graphs indicate that the test as made under the usual standard conditions is not a good index of the ductile properties of the individual asphalts.

In 1927, W. F. Smith,[6] of the Oklahoma State Highway Commission, proposed that the ductility test be made on filler-grade asphalts to eliminate materials that were extremely brittle at low temperatures. He suggested that the test be made at a rate of pull of 5 centimeters per minute at a temperature of 32° F. Under these conditions, however, many of the test specimens were broken at the start of the test. For many years the California Division of Highways specified a ductility test on 95+ road oil to be made at 34°–35° F. and at a rate of pull of 5 centimeters per minute. No difficulty in testing was experienced with this type of material since it was relatively soft at the test temperature. Tests at 34°–35° F. and at the rate of 5 centimeters a minute were made for many years on asphaltic residues of 80 and 100 penetration, but slightly lower penetrations of the residues at 77° F. often resulted in the breaking rather than in the elongation of the specimens.

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[6] Low Temperature Ductility of Filler Grade Asphalts. A. S. T. M. Proceedings, vol. 27, pt. II. 1927.

In more recent years, the rate of pull of 5 centimeters per minute and temperature of test of 32° F. have been suggested for testing asphalt cements, but the failure of such a large number of materials to pull without snapping has resulted in increasing the temperature slightly, in decreasing the rate of pull, and in some cases doing both. The decrease in the rate of pull is intended to reduce the initial shock to which the specimens are subjected when the test is started.

Tests made at 39.2° F., 5 centimeters a minute, to meet the specification limit for ductility of one-tenth the penetration minus 1 or one-tenth the penetration have been criticised by W. H. Flood[7] because of the small range in values obtained, the assumed relationship between ductility and penetration, and the difficulty of maintaining a constant temperature during the test. He has suggested that a ductility test at 60° F. at a rate of pull of 5 centimeters a minute would be preferable to the low temperature test or to the normal test at 77° F. which, in most cases, gives high ductilities of little values for comparative purposes. However, the data given in table 13, and shown in figures 9 and 10, indicate that there would be considerable difficulty in designating an intermediate temperature and specifying test limits that would be adequate for evaluating the essential characteristics of asphalts intended for various uses.

Ductility at low temperatures.—All of the asphalts were tested for ductility at 39.2° F. with rates of pull of 5, 1, and ¼ centimeter per minute. Tests were also made at 32° F. with rates of pull of 1, and ¼ centimeter per minute. The results of these tests are given in table 12.

As can be seen from this table, a ductility requirement at 39.2° F., 5 centimeters per minute, of one-tenth the penetration is highly restrictive. None of the asphalts of the 50–60 grade and only one of the 85–100 penetration grade would meet this requirement. A requirement of one-tenth the penetration minus 1 would permit only three of the 85–100 penetration materials to pass and none of the 50–60 asphalts.

Although a speed of 1 centimeter per minute has had little use as a specification requirement for ductility tests made at 39.2° F. or 32° F., the ductility of both grades of asphalt at this rate of pull and at these two temperatures are reported for comparative purposes. At 32° F. and a speed of 1 centimeter per minute, a requirement of one-tenth the penetration at 77° F. would eliminate all asphalts of both grades and a requirement of one-tenth the penetration minus 1 would eliminate all but one 50–60 and three 85–100 penetration asphalts. At 39.2° F. and a speed of 1 centimeter per minute, a requirement of one-tenth the penetration at 77° F. would eliminate 26 of the 50–60 and 11 of the 85–100 grades, and for a requirement of one-tenth the penetration minus 1, nine 50–60 and eight 85–100 asphalts would fail.

DUCTILITIES DEPENDENT ON BASE PETROLEUM AND PROCESSING

A rate of pull of ¼ centimeter per minute has been used at temperatures of both 32° and 39.2° F. Requirements of one-tenth the penetration at 77° F., and one-tenth the penetration minus 1 have been used where 32° F. has been designated as the test temperature. For the test temperature of 39.2° F. a ductility requirement of one-tenth the penetration at 77° F. has been used.

Of the 50–60 penetration asphalts tested at 39.2° F., ¼ centimeter per minute, there were five that failed to meet the requirement of one-tenth the penetration at 77° F. Four of these were within 1 centimeter of the required ductility. The ductility of sample 39 was 2.7 centimeters below its required minimum. Of the 85–100 penetration asphalts, there were three materials that failed to meet this requirement. These were samples 24, 33, and 36 which were 1.6, 0.8, and 2.2 centimeters, respectively, below the calculated minimum requirements.

When the ductility is determined at 32° F., ¼ centimeter per minute, a specification requirement of either one-tenth the penetration at 77° F. or one-tenth the penetration at 77° F. minus 1 becomes highly restrictive. With a decrease of only 7.2° F. in test temperature from 39.2° to 32° F., 33 asphalts of the 50–60 grade failed to meet a requirement of one-tenth the penetration as compared to only 5 that failed when tested at 39.2° F. With the test temperature at 32° F., 22 failed to meet the requirement of one-tenth the penetration minus 1. This lower test temperature would cause the rejection of the California asphalts which showed unusually high ductility at 39.2° F. as well as a majority of the mid-continent asphalts which were slightly above the minimum test requirement for ductility at this rate of pull at the slightly higher temperature.

Of the 85–100 penetration asphalts, 18 would fail to meet a requirement of one-tenth the penetration as compared to only 3 that failed when tested at 39.2° F. For this grade, 14 materials failed to meet the requirement of one-tenth the penetration minus 1. Seventeen of the 18 asphalts of the 85–100 grade that failed to have a ductility equal to one-tenth the penetration were from the same source as the 50–60 asphalts that would not meet a similar requirement. Ten of the fourteen materials of the 85–100 grade that failed to meet a requirement of one-tenth the penetration minus 1 were also from the same source as the 50–60 asphalts failing to meet a similar requirement.

One of the most interesting points in this ductility study is the behavior of four of the five California products, samples 1, 2, 3, and 4. The samples of the 85–100 grade had extremely high ductilities at 39.2° and 32° F. at a rate of pull of ¼ centimeter per minute, and at 39.2° F. when pulled at 1 centimeter a minute. At 32° F., 1 centimeter per minute, the materials would not elongate. The 50–60 grades of the California materials had slightly higher values than any of the other samples when pulled at 1 centimeter per minute at 39.2° F., and they were many times more ductile when pulled at ¼ centimeter a minute. At 32° F., they broke when pulled at a speed of ¼ or 1 centimeter per minute. Both grades of sample 5, which had a ductility at 77° F. approximately the same as the other four California asphalts, showed a decidedly different behavior than the others when tested at ¼ and 1 centimeter a minute at 39.2° F., and the 85–100 sample showed a difference when tested at ¼ centimeter a minute at 32° F.

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[7] Ductility at Low Temperatures. Proceedings of the Association of Asphalt Paving Technologists. January 1935.

In these low-temperature tests there is apparently no relation between the ductility, the temperature, or the rate of pull that is common to all the asphalts or to the asphalts from a particular source. The characteristics shown by these ductility tests are probably due to the properties of the base petroleum and processing of the particular asphalts.

TOUGHNESS TEST USED TO MEASURE BRITTLENESS AT LOW TEMPERATURE

Toughness Test.—The toughness test for asphalt cements was first used by the New York Department of Public Works in 1911 as a specification requirement for the control of asphalts for asphalt-bound macadam.[8]

The 1932 specifications of the department contained a clause that required 50–65 and 85–100 penetration asphalts to have a minimum toughness of 10 and 100–110 penetration asphalt to have a minimum toughness of 15. The test is now made on a cylinder of asphalt 1¾ inches in diameter and 1¾ inches high. The test procedure is as follows:

The material to be tested is melted at the lowest possible temperature and poured into a specially constructed split mold which has previously been amalgamated. The filled mold is allowed to cool to room temperature and then chilled to 32° F. (0° C.). The excess material is then cut from the top by a heated knife and the specimens removed from the mold. These are then maintained at 32° F. (0° C.) for 1 hour before testing.

The specimens are tested in a Page impact machine, which is the same as used in the Standard Method of Testing for Toughness of Rock, A. S. T. M. Specification D 3–18. A special bath is so constructed that it rests flat on the anvil of the Page impact machine and is braced by three legs that rest upon the base. The bath is of such depth that when filled with ice and water the specimen is completely covered, assuring a temperature of 32° F. (0° C.) during the test. A patch of paper approximately 1 inch square is placed between the plunger of the impact machine and the cylinder of asphalt being tested. In testing bituminous materials the 2-kilogram hammer is allowed to drop 5 centimeters onto the intervening plunger for the first blow and the drop is then increased 5 centimeters for each succeeding blow until rupture of the specimen occurs. The height from which the hammer falls when rupture occurs is the numerical value for the toughness of the material under test.

The toughness test values of the 50–60 and 85–100 penetration asphalts are given in table 12. The results shown are the average values for two tests. These values ranged from 5 to 22.5 with an average of 11.7 for the 50–60 grade and from 5 to 35 with an average of 13.1 for the 85–100 grade. Based on the minimum requirement of 10 as used in the New York State specification, there were 9 samples of the 50–60 and 5 samples of the 85–100 penetration asphalt that would not pass the test. A comparison of the results for the two penetration grades shows that 13 asphalts of the 85–100 grade had lower and 19 had higher values for toughness than asphalts from the same source in the 50–60 grade. The test was devised to identify and restrict the use of the type of asphalts that are brittle at low temperature. It may be noted that asphalts of the same consistency at the test temperature of 32° F., as measured by the penetration test, vary greatly in toughness.

In table 15, the results of tests that are essentially identification tests of source or processing are given for both grades of asphalts. Tests for solubility in 86° B. naphtha, the solubility in ethyl ether, and the fixed carbon content, while not in general use, are still required by a few specifications. The determination of the percentage of sulphur has been included in some specifications to control more definitely the source of the base petroleum. The Oliensis spot test has been used to indicate the presence of cracked materials, and the film test has been proposed as an accelerated test of the weather-resistant property of asphaltic materials. A brief discussion of these tests will be made and the effect of the proposed requirements on the rejection or acceptance of the asphalts in both grades will be noted.

SPECIAL TESTS USED TO IDENTIFY SOURCE OR PROCESSING

Solubility in 86° B. naphtha.—Many of the specifications of earlier years contained clauses limiting the percentage of bitumen insoluble in 86° B. naphtha, and the fixed carbon in asphaltic materials. These constituents are dependent on both the base petroleum and the processing. While they may have value as identification tests and in controlling uniformity of supply, the much greater number of base petroleums now in use, together with the present wide variation in refining processes, have somewhat impaired their usefulness for these purposes.

Only one State specification for 50–60 asphalt carries a requirement for the percentage of bitumen insoluble in 86° B. naphtha. The limits set by this State of 15 to 29 percent would cause the rejection of the five California asphalts that were low in asphaltenes, and five other materials that had values over the maximum limit. For the 85–100 penetration grade, the same limits would also reject the five California asphalts, the Colombian asphalt, and two mid-continent asphalts that had similar characteristics. These eight 85–100 materials had a low percentage of asphaltenes.

Fixed carbon.—Only one State has employed the fixed carbon test for the purpose of controlling asphalt cements. The requirement stipulated that the bitumen should show between 8 and 17 percent fixed carbon. Four of the five California asphalts of both grades had less than 8 percent fixed carbon and were the only materials failing to meet this specification. For asphalts from the same source, the percentage of asphaltenes tended to increase as the percentage of fixed carbon increased. As indicated by figure 13, however, there was no definite relationship between the fixed carbon and the organic matter insoluble in 86° B. naphtha for these materials as a group.

Solubility in ethyl ether.—One of the reasons for the discontinuance of the test for solubility in 86° B. naphtha was the inability to obtain a uniform supply of the naphtha. One State has substituted ethyl ether which is a definite chemical compound having uniform and unchanging properties. The requirement of this specification for solubility in ethyl ether limits the percentage of insoluble matter to a maximum of 25 percent. Table 15 shows that all of the asphalts in this study had less than 25 percent insoluble matter.

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[8] Method of Determining the Toughness of Bituminous Materials. J. E. Meyers, Engineering Record, vol. 67, No 3, January 18, 1913.

Table 15.—*Chemical and special tests on the 50–60 and 85–100 penetration asphalts*

Identification No.	50–60 penetration asphalts									85–100 penetration asphalts								
					Oliensis test			Film test						Oliensis test			Film test	
	Organic matter insoluble in ethyl ether	Organic matter insoluble in 86° B. naphtha	Fixed carbon	Sulphur	Character of spot	Gilsonite equivalent	Xylene equivalent	Condition after 5 hours at 325° F.	Condition after 24 hours at 325° F.	Organic matter insoluble in ethyl ether	Organic matter insoluble in 86° B. naphtha	Fixed carbon	Sulphur	Character of spot	Gilsonite equivalent	Xylene equivalent	Condition after 5 hours at 325° F.	Condition after 24 hours at 325° F.
	Pct.	*Pct.*	*Pct.*	*Pct.*						*Pct.*	*Pct.*	*Pct.*	*Pct.*					
1	3.3	10.7	7.4	1.04	Negative			Clear	Clear	3.0	10.5	7.0	0.91	Negative			Clear	Clear.
2	5.5	10.6	7.4	1.30	do			do	do	3.9	10.1	7.3	1.27	do			do	Do.
3	4.3	10.0	6.9	1.25	do			do	do	3.8	9.9	6.8	1.29	do			do	Do.
4	4.3	11.8	6.5	1.38	do			do	do	4.4	11.5	6.6	1.22	do			do	Do.
5	5.6	12.6	8.9	1.03	do			do	do	6.5	14.7	9.0	2.14	Positive	0–1	0–2	do	Do.
6	11.3	18.1	12.4	1.64	do			do	do	9.4	14.3	11.6	1.48	Negative			do	Do.
7	23.9	28.2	11.3	5.96	do			do	do	20.7	26.4	11.4	5.32	do			do	Do.
8	23.4	30.9	14.8	5.24	Positive	50–60	20–24	do	do	21.8	28.0	14.9	4.79	Positive	50–60	24–28	do	Do.
9	22.3	29.3	13.9	6.20	Negative			do	do	20.2	26.2	13.6	5.65	Negative			do	Do.
10	21.8	28.8	11.7	6.00	do			do	do	20.9	26.4	11.9	5.71	do			do	Do.
11	22.7	28.1	11.3	5.87	do			do	do	20.2	26.4	11.5	5.71	do			do	Do.
12	22.0	28.0	13.1	6.09	do			do	do	19.6	25.5	12.3	5.36	do			do	Do.
13	20.4	27.5	16.0	3.14	Positive	60–70	28–32	do	do	17.4	23.1	14.2	2.93	Positive	5–8	2–4	do	Do.
14	15.6	21.7	13.0	3.21	Negative			do	do	14.4	20.4	12.3	2.95	Negative			do	Do.
15	17.8	24.8	14.9	3.27	Positive	40–50	24–28	do	do	15.2	21.7	13.2	3.27	Positive	5–10	2–4	do	Do.
16	19.0	25.6	14.6	3.24	do	40–50	28–32	do	do	15.9	22.0	13.3	2.91	do	5–10	2–4	do	Do.
17	15.7	22.9	13.0	3.21	Negative			do	do	14.1	18.0	11.6	2.72	Negative			do	Do.
18	18.5	24.8	13.8	3.94	do			do	do	18.9	24.6	13.0	4.93	do			do	Do.
19	14.3	19.7	14.7	3.81	do			do	do	13.2	19.0	13.5	3.43	Positive	0–1	0–2	do	Do.
20	20.9	30.8	13.4	3.46	do			do	do	18.9	25.4	12.5	3.30	Negative			do	Do.
21	16.7	24.2	13.9	3.66	do			do	do	14.2	20.2	13.7	3.46	do			do	Do.
22	19.5	27.9	14.1	3.20	do			do	do	16.0	21.4	13.3	3.25	do			do	Do.
23	11.6	21.6	12.8	.84	Positive	60–70	84–88	Coagulated	Coagulated	9.4	15.9	11.7	.98	Positive	30–35	44–48	Coagulated	Coagulated.
24	13.0	20.4	11.9	1.06	Negative			Clear	Clear	10.3	16.6	10.9	1.04	Negative			Clear	Clear.
25	11.0	17.3	11.0	1.07	do			do	do	8.7	12.7	10.4	1.13	do			do	Do.
26	15.3	25.4	12.3	1.04	Positive	30–40	16–20	do	do	11.6	18.2	11.2	.89	Positive	2–4	2–4	do	Do.
27	11.6	19.3	12.3	1.10	Negative			do	do	7.7	13.2	10.7	.95	Negative			do	Do.
28	12.9	23.0	12.2	.95	Positive	40–50	32–36	do	do	8.3	15.1	11.4	.87	Positive	5–10	4–8	do	Do.
29										9.9	18.5	12.9	1.53	do	60–80	[1]56–60	Coagulated	Coagulated.
30	15.7	23.3	12.6	.72	Negative			Clear	Clear	11.9	18.3	12.2	.54	Negative			Clear	Clear.
31	24.2	30.2	13.6	3.93	do			do	do	21.5	27.7	13.2	3.34	do			do	Do.
32	15.1	21.4	15.4	.86	do			do	do	13.8	18.9	14.3	.88	do			do	Do.
33	15.5	24.5	16.3	.74	Positive	70–80	[1]60–64	Coagulated	Coagulated	14.4	22.5	14.9	.80	Positive	80–90	[1]60–64	Coagulated	Coagulated.
34	20.5	28.5	16.6	3.97	do	50–60	28–32	Clear	Clear	17.0	26.3	15.8	3.81	do	15–25	12–16	Clear	Clear.
35	9.5	19.1	10.8	2.64	Negative			do	do	10.3	18.7	9.6	2.41	Negative			do	Do.
36	11.1	20.9	10.8	2.52	Positive	0.5	0–4	do	do	14.1	21.2	12.6	.94	Positive	0–2	0–2	do	Do.
37	19.3	27.0	11.3	3.18	do	0–5	0–4	do	do	16.0	24.7	10.7	2.83	Negative			do	Do.
38	18.5	25.7	11.8	2.08	do	0–5	0–4	do	do	15.9	23.6	10.9	1.69	do			do	Do.
39	12.1	23.0	14.3	3.55	do	60–70	24–28	do	do	10.7	19.9	12.8	3.31	Positive	35–40	16–20	do	Do.
40	18.6	31.9	15.0	.44	do	90+	80–84	do	Coagulated	16.5	25.8	14.5	.47	do	90+	80–84	do	Coagulated.

[1] Maximum value same as spot with 100 percent xylene.

Figure 14 indicates an approximately straight line relationship between the percentage of organic matter insoluble in ether and the percentage of organic matter insoluble in 86° B. naphtha for both grades of asphalt.

Sulphur content.—Although a number of specifications have carried a requirement that the sulphur content of asphalt cements should be more than 3 percent, no logical reason has been advanced for such a provision. Victor Nicholson[9] stated that in 1915 the Bureau of Streets of Chicago found it was possible to obtain inferior pavements with some asphalts meeting the specifications of the American Association for the Standardization of Paving Specifications. He added that the shortcomings in these specifications were overcome by altering several clauses and by the insertion of a requirement that the sulphur content should be not less than 3 percent.

In his discussion of the sulphur requirement, no direct evidence was offered by Nicholson to show the superiority of pavements containing asphalts of high sulphur content. He did attempt, however, to correlate the sulphur content with other physical properties of the materials. He concluded that the sulphur content has considerable influence on the viscosity of the asphalts as indicated by the values of the fluidity factor and float test. He stated that the consistency, as represented by the fluidity factor, did not vary directly with the sulphur content, but that asphalts having over 3 percent sulphur had fluidity factors over 130.

The fluidity factor can be altered to a considerable extent by blowing and blending, and the observations of Nicholson are only partially corroborated by the sulphur determinations made on the asphalts of this investigation, the data for which are shown in table 15.

Nineteen materials of the 50–60 grade had sulphur contents of less than 3 percent, and 5 of these had fluidity factors of more than 130. Twenty materials had more than 3 percent sulphur and 7 of these had fluidity factors of less than 130. In the 85–100 penetration grade, 25 of the materials had sulphur contents lower than 3 percent, and in no case was the fluidity factor over 130. Nine of the 15 asphalts having more than 3 percent sulphur had fluidity factors less than 130.

In 29 cases, the asphalts of the 50–60 grade had higher percentages of sulphur than the corresponding asphalts of the 85–100 grade. The 85–100 grade of samples 5 and 18 were the only materials that had appreciably higher sulphur contents than the 50–60 asphalts from the same source. Both grades of sample 35 and the 50–60 grade of sample 36, had essentially the same sulphur content, yet the 85–100 grade of sample 36 had a decidedly lower sulphur content. These asphalts were from the same producer, who used crude petroleum from several sources; and the sulphur contents, as well as the other test data, indicate that the two grades of sample 36 were refined from different base petroleums.

Nineteen of the 50–60 asphalts and 25 of the 85–100

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[9] The Sulphur Requirement, Proceedings of Association of Asphalt Paving Technologists. January 1935.

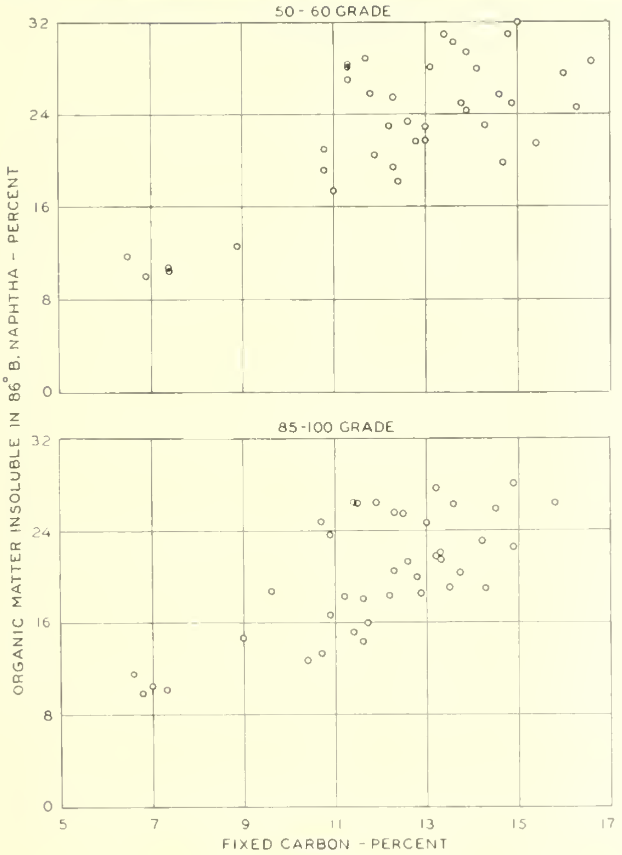

FIGURE 13.—RELATION BETWEEN FIXED CARBON AND ORGANIC MATTER INSOLUBLE IN 86° B. NAPHTHA OF THE 50–60 AND 85–100 PENETRATION ASPHALTS.

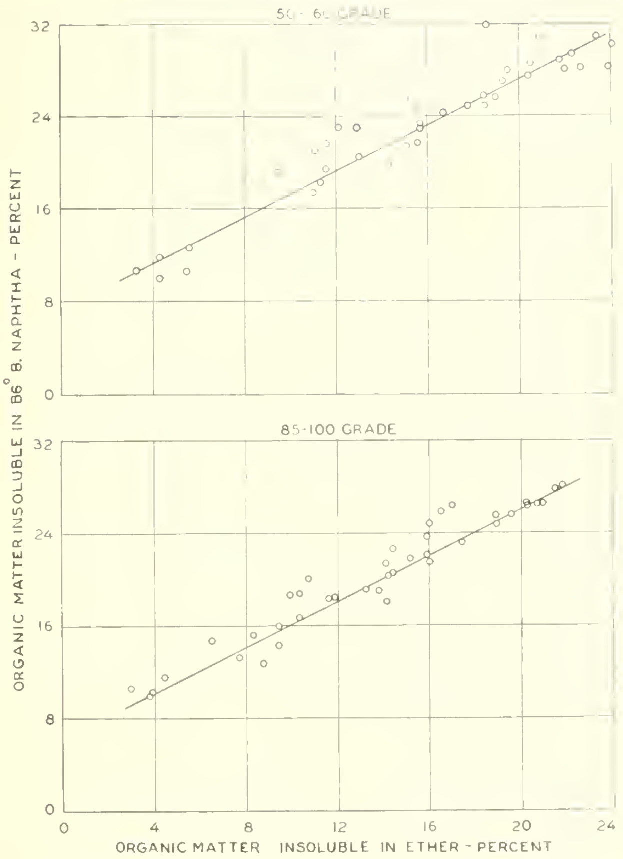

FIGURE 14.—RELATION BETWEEN ORGANIC MATTER INSOLUBLE IN ETHER AND ORGANIC MATTER INSOLUBLE IN 86° B. NAPHTHA OF THE 50–60 AND 85–100 PENETRATION ASPHALTS.

penetration asphalts would fail to meet a specification requirement of 3 percent or more. Although there are California asphalts of high sulphur contents, those represented in this study would not meet this requirement. Many mid-continent products would also be rejected. While it is generally considered that the sulphur requirement was introduced in specifications to insure the use of Mexican asphalts, a minimum limit of 3 percent will permit the use of asphalts refined from Venezuelan, Arkansas, Texas, and Wyoming petroleums. However, as shown in figure 15, a sulphur requirement is essentially an identification test of source.

SEVEN OF THE ASPHALTS HAD XYLENE EQUIVALENTS OVER 40

Reaction to Oliensis test.—The Oliensis spot test [10] has been used for several years as a means for determining if asphaltic materials have been overheated or cracked during the refining process. As initially proposed, the test served only as a means to identify overheated or cracked materials. In 1936,[11] the test was improved so that, within certain limits, a quantitative measure of the degree of heterogeneity may be determined.

Since the development of the Oliensis test several States have included a requirement for a spot test in their specifications for asphaltic materials. In order to determine the reaction of these asphalts to the Oliensis spot test, they were all tested qualitatively and those that gave positive spots were tested with both gilsonite and xylene, as homogenizers. The results of these tests for both the 50–60 and 85–100 penetration asphalts are given in table 15. This table gives the character of the spot, and the gilsonite and xylene equivalents of those samples showing a positive spot.

There were 14 asphalts of the 50–60 grade and 15 asphalts of the 85–100 grade that gave positive spots with the standard naphtha. Both grades of the asphalts from the same source gave similar spots in most instances, although the 50–60 grades of samples 37 and 38 were positive and the 85–100 grades were negative. In the two grades of samples 5 and 19, the asphalts of 85–100 grade only were positive. Sample 29 was not represented in the 50–60 asphalts.

A heterogeneous asphalt, one giving a positive spot, may be made to appear homogeneous, having a negative spot, by the use of various homogenizers.[11] As indicated in table 15, the addition of relatively small amounts of gilsonite in some instances will make the asphaltic material show a negative spot. In other cases, the quantity of gilsonite necessary to produce a blend showing a negative spot is far too great to make its use profitable. However, the addition of homogeneous asphaltic materials to mask the presence of heterogeneous materials has been noted, and such blends obviously defeat the purpose of the Oliensis test.

The solvent xylene, when used as a homogenizer in varying proportion with the standard naphtha, gives a measure of the degree of heterogeneity of asphaltic materials. The values for xylene equivalents in table 15 can be considered as indicating the relative heterogeneity of these materials.

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[10] A Quality Test for Determining the Degree of Heterogeneity of Asphalts, by G. L. Oliensis, Proceedings, A. S. T. M., vol. 33, pt. 2, 1933.

[11] A Further Study of the Heterogeneity of Asphalt—A Quantitative Method. By G. L. Oliensis, Proceedings, A. S. T. M., vol. 36, pt. 2, 1936.

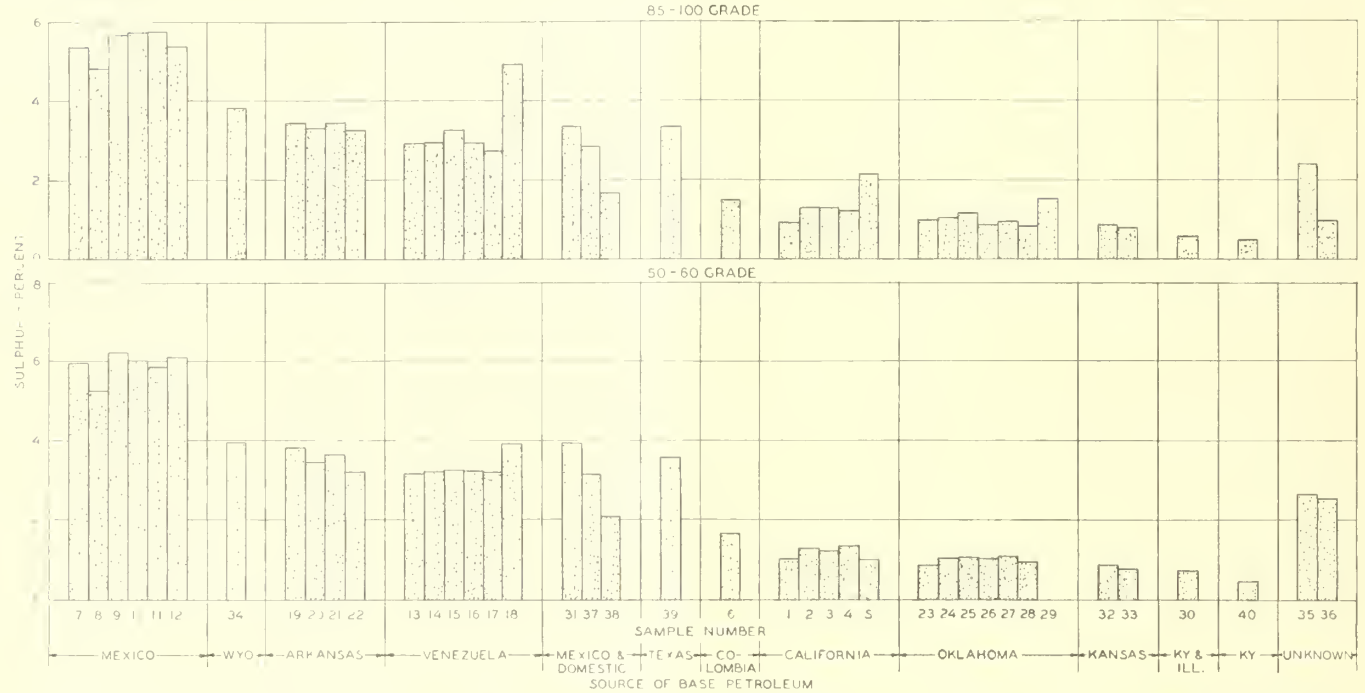

Figure 15.—The Source of the Base Petroleum in Relation to the Sulphur Content of the 50–60 and 85–100 Penetration Asphalts.

Of the 14 asphalts of the 50–60 penetration grade that were heterogeneous, 3 had xylene equivalents of less than 16 and 3 that were greater than 60. Of the 15 heterogeneous asphalts of the 85–100 penetration grade, nine had xylene equivalents of less than 16 and two were higher than 60. Samples 33 of the 50–60 grade and 29 and 33 of the 85–100 grade gave positive reactions with 100 percent xylene, but the maximum values reported are the minimum amounts of xylene in the xylene-naphtha solvent that gave a spot of the same intensity as when xylene alone was used as solvent.

A comparison of the asphalts of the two grades shows that 11 asphalts of the 50–60 grade had higher xylene equivalents than the corresponding asphalts of the 85–100 penetration grade. This would seem to indicate that under uniform refining conditions the development of heterogeneity in the residual asphalt progresses as the material becomes harder. Table 16 shows the distribution of the materials having positive spots according to the process used in manufacture.

Film test.—In 1937, J. R. Benson, of the State Highway Commission of Kansas presented two papers,[12] [13] describing the possibilities of the so-called film test as an accelerated weathering test for asphaltic materials. In these papers the author showed the reaction of translucent asphaltic films approximately 0.001 inch thick when exposed to six artificial weathering conditions. The physical changes due to the various types of exposure were determined by direct observation under the microscope. The test procedure is as follows:

The film of asphalt to be tested is spread on a 1- by 3-inch glass microscope slide by means of a special gage so constructed as to give a film approximately 0.001 inch thick. The slide is then placed horizontally in a constant-temperature oven maintained at 325° F. After 5 hours the slide is removed from the oven, cooled to room temperature, and examined under a microscope at a magnification between 200 and 360 under transmitted light. If the film is clear the slide is again placed in the oven at 325° F. and exposed for a total of 24 hours.

Table 16.—*The refining process in relation to the xylene equivalents of the heterogeneous asphalts*

Sample No.	Xylene equivalent		Refining process
	50–60 grade	85–100 grade	
5	------	0–2	Vacuum distillation.
8	20–24	24–28	Unknown.
13	28–32	2–4	Fire and steam distillation.
15	24–28	2–4	Steam distillation.
16	28–32	2–4	Do.
19	------	0–2	Vacuum distillation.
23	84–88	44–48	Unknown.
26	16–20	2–4	Vacuum distillation and oxidation.
28	32–36	4–8	Unknown.
29	------	[1] 56–60	Do.
33	[1] 60–64	[1] 60–64	Cracking process.
34	28–32	12–16	Fire and steam distillation.
36	0–4	0–2	Do.
37	0–4	------	Unknown.
38	0–4	------	Do.
39	24–28	16–20	Do.
40	80–84	80–84	Cracking process.

[1] Maximum value same as spot with 100 percent xylene.

ONLY ASPHALTS HAVING HIGH XYLENE EQUIVALENTS FAILED TO PASS FILM TEST

The condition of the film after the 5- and 24-hour periods is determined as either clear or coagulated. A film is said to be clear when it presents a smooth surface and shows no change in condition other than hardening.

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[12] A Study of Translucent Asphaltic Films, Proceedings, Highway Research Board, vol. 17, December 1937.

[13] Microscopic Reactions in Translucent Asphaltic Films. Proceedings of the Association of Asphalt Paving Technologists. vol. 9, December 1937.

A film is said to be coagulated when it develops a lacy or granular structure, with the asphalt often appearing to have separated into two distinct components, one light and the other dark.

Although the work done by Benson has been largely on materials of the liquid asphaltic type, it was decided to observe the behavior of the various asphalt cements of this investigation when subjected to the film test as required in the Kansas specification. The results of film test for the 50–60 and 85–100 penetration asphalts are given in table 15. Of the 50–60 grade, samples 23 and 33 showed definite coagulation at the end of the 5-hour and 24-hour periods. Of the 85–100 grade these same asphalts, together with sample 29, also showed coagulation at the end of the 5-hour and 24-hour test periods. Sample 40 in both grades showed coagulation at 24 hours but not at 5 hours. These seven asphalts showed the highest degree of heterogeneity as rated by the Oliensis test, all having xylene equivalents in excess of 44.

Table 17 shows the asphalts that failed to meet the special requirements that have been discussed. It indicates that, in general, the 85–100 penetration asphalts fail in a lesser number of tests than the 50–60 asphalts. The 85–100 asphalts could not be tested for penetration under normal test procedure at 115° F., 50 grams, 5 seconds, so that the susceptibility factor

$$\frac{\text{penetration } 115^\circ \text{ F., 50 gm., 5 sec.} - \text{penetration } 32^\circ \text{ F., 200 gm., 60 sec.}}{\text{penetration } 77^\circ \text{ F., 100 gm., 5 sec.}}$$

was not determined for this grade. The float-test index was specified only for asphalts having penetrations at 77° F. of from 40 to 70 and the designated specification requirement is not applicable to the 85–100 penetration asphalts.

TABLE 17.—*Identification [1] of the 50–60 and 85–100 penetration asphalts which fail to meet special test requirements*

Special test	Fluidity factor		Float-test index		Pen. 39.2° F. / Pen. 77° F. [2]		Pen. 115° F.—Pen. 32° F. / Pen. 77° F. [2]		Ductility at 39.2° F., ¼ cm. per minute		Ductility at 32° F., ¼ cm. per minute				Ductility at 39.2° F., 5 cm. per minute		Toughness test		Organic matter insoluble in 86° B. naphtha		Fixed carbon		Sulphur		Film test		Oliensis test		Number of tests each sample fails to pass	
Proposed test requirement	140+		90+		30+ percent		4.2−		1/10 pen. at 77° F.		1/10 pen. at 77° F. −1		1/10 pen. at 77° F.		1/10 pen. at 77° F.		10+		15–29 percent		8–17 percent		3.0+ percent		Shall not coagulate		Shall show negative spot			
Penetration grade	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100	50 to 60	85 to 100
Sample number:																														
1	x	x	x	x	x	x	x	(3)	---	---	x	---	x	---	x	x	x	---	x	x	x	x	x	x	---	---	---	---	11	7
2	x	x	x	---	x	---	x	---	---	---	x	---	x	---	x	x	x	---	x	x	x	x	x	x	---	---	---	---	11	5
3	x	x	x	---	x	x	x	---	---	---	x	---	x	---	x	x	x	---	x	x	x	x	x	x	---	---	---	---	11	6
4	x	x	x	---	x	x	x	---	---	---	x	---	x	---	x	x	x	---	x	x	x	x	x	x	---	---	---	---	11	6
5	x	x	x	---	---	x	x	---	---	---	x	---	x	x	x	x	x	---	x	x	---	---	x	x	---	---	---	x	9	7
6	x	x	x	---	---	---	x	---	---	---	---	---	x	---	x	x	---	x	---	x	---	---	x	x	---	---	---	---	6	5
7	---	---	---	---	---	---	x	---	---	---	---	---	---	---	x	---	---	---	---	---	---	---	---	---	---	---	---	---	2	0
8	x	x	---	---	---	---	x	---	---	---	x	x	x	x	x	x	---	---	x	---	---	---	---	---	---	---	x	x	7	5
9	---	---	---	---	---	---	x	---	---	---	---	---	---	---	x	x	---	---	x	---	---	---	---	---	---	---	---	---	3	1
10	---	---	---	---	---	---	x	---	---	---	---	---	---	---	x	x	---	---	---	---	---	---	---	---	---	---	---	---	2	1
11	---	---	---	---	---	---	---	---	---	---	---	---	x	---	x	x	---	---	---	---	---	---	---	---	---	---	---	---	2	1
12	---	---	---	---	---	---	---	---	---	---	---	---	---	---	x	x	---	---	---	---	---	---	---	---	---	---	---	---	1	1
13	x	x	---	---	---	---	---	---	---	---	---	---	x	---	x	x	---	---	---	---	---	---	---	x	---	---	x	x	4	4
14	x	x	x	---	---	---	x	---	---	---	---	---	x	---	x	x	---	---	---	---	---	---	---	x	---	---	---	---	5	3
15	x	x	x	---	---	---	x	---	---	---	---	---	x	---	x	x	---	---	---	---	---	---	---	---	---	---	x	x	6	3
16	x	x	x	---	---	---	---	---	---	---	---	---	x	---	x	x	---	---	---	---	---	---	---	x	---	---	x	x	5	4
17	x	x	x	---	---	---	x	---	---	---	---	---	---	---	x	x	---	---	---	---	---	---	---	x	---	---	---	---	4	3
18	---	---	x	---	---	---	x	---	---	---	---	---	---	---	x	x	---	---	---	---	---	---	---	---	---	---	---	---	3	1
19	---	x	x	---	---	---	x	---	---	---	x	---	x	x	x	x	---	x	---	---	---	---	---	---	---	---	---	x	5	5
20	---	x	---	---	---	---	---	---	x	---	x	x	x	x	x	x	---	---	x	---	---	---	---	---	---	---	---	---	5	4
21	---	x	---	---	---	---	x	---	---	---	x	---	x	---	x	x	x	---	---	---	---	---	---	---	---	---	---	---	5	2
22	---	x	---	---	---	---	---	---	---	---	x	---	x	---	x	x	---	---	---	---	---	---	---	---	---	---	---	---	3	2
23	x	x	x	---	---	x	x	---	x	---	x	x	x	x	x	x	---	---	---	---	---	---	x	x	x	x	x	x	10	8
24	---	x	---	---	---	---	---	---	x	x	x	x	x	x	x	---	x	---	---	---	---	---	x	x	---	---	---	---	6	5
25	---	x	x	---	---	---	x	---	---	---	x	x	x	x	x	x	x	x	---	x	---	---	x	x	---	---	---	---	7	7
26	x	x	x	---	---	---	x	---	---	---	x	---	x	x	x	x	---	---	---	---	---	---	x	x	---	---	x	x	8	5
27	x	x	x	---	---	x	x	---	---	---	---	---	x	---	x	x	---	---	---	x	---	---	x	x	---	---	---	---	6	5
28	---	x	x	---	---	x	x	---	---	---	x	---	x	---	x	x	---	x	---	---	---	---	x	x	---	---	x	x	7	6
29	---	x	---	---	---	x	---	---	---	---	---	x	---	x	---	x	---	x	---	---	---	---	---	x	---	x	---	x	---	9
30	---	x	x	---	---	---	---	---	---	---	---	x	x	x	x	x	---	---	---	---	---	---	x	x	---	---	---	---	4	5
31	---	x	---	---	---	---	---	---	x	---	x	x	x	x	x	x	---	---	x	---	---	---	---	---	---	---	---	---	5	4
32	x	x	x	---	---	---	x	---	---	---	x	x	x	x	x	x	---	---	---	---	---	---	x	x	---	---	---	---	7	5
33	x	x	x	---	---	---	x	---	---	x	x	x	x	x	x	x	---	---	---	---	---	---	x	x	x	x	x	x	9	8
34	x	x	---	---	---	---	---	---	---	---	x	x	x	x	x	x	---	---	---	---	---	---	---	---	---	---	x	x	5	5
35	x	x	x	---	---	---	x	---	---	---	x	---	x	---	x	x	---	---	---	---	---	---	x	x	---	---	---	---	7	3
36	x	x	x	---	---	---	x	---	---	x	---	x	x	x	x	x	---	---	---	---	---	---	x	x	---	---	x	x	7	7
37	---	x	---	---	---	---	---	---	---	---	---	---	x	x	x	x	---	---	---	---	---	---	---	x	---	---	x	---	3	4
38	---	x	---	---	---	---	---	---	---	---	---	x	x	x	x	x	---	---	---	---	---	---	x	x	---	---	x	---	4	5
39	x	x	x	---	---	x	x	---	x	---	x	x	x	x	x	x	x	---	---	---	---	---	---	---	---	---	x	x	9	6
40	x	x	x	---	---	---	x	---	---	---	x	---	x	---	x	x	---	---	x	---	---	---	x	x	x	x	x	x	10	5
Number of samples failing each test	22	34	24	1	4	9	27	---	5	3	22	14	33	18	39	38	9	5	10	8	4	4	19	25	3	4	14	15		

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[1] Samples marked (x) fail to meet requirement.

[2] Penetrations at various temperatures as follows:
32° F., 200 gm., 60 sec.
39.2° F., 200 gm., 60 sec.
77° F., 100 gm., 5 sec.
115° F., 50 gm., 5 sec.

[3] Penetrations not made at 115° F. on 85-100 penetration asphalts.

Excluding these two requirements, in 6 out of 11 cases the 50–60 asphalts had a larger number of samples failing than did the 85–100 asphalts. The 85–100 asphalts showed the higher number of failures for the fluidity factor, the susceptibility factor using penetration at 77° F., 100 grams 5 seconds and 39.2° F., 200 grams 60 seconds, the sulphur content, and the reaction to the film and the Oliensis test. Both grades showed the same number of failures for the fixed carbon test.

For the 50–60 asphalts, the requirements for ductility at 39.2° F., 5 centimeters a minute, and ductility at 32° F., ¼ centimeter a minute, of one-tenth the penetration, the susceptibility factor involving tests at 3 temperatures, 115°, 77°, and 32° F., the float-test index, and the fluidity factor, in the order named, were the most restrictive. The requirements for the film test, fixed carbon, the susceptibility factor using the temperatures 39.2° and 77° F., and the ductility at 39.2° F., ¼ centimeter per minute of one-tenth the penetration, were the least restrictive.

For the 85–100 penetration asphalts the requirements for ductility at 39.2° F., 5 centimeters a minute of one-tenth the penetration, the fluidity factor, the sulphur content, and ductility at 32° F., ¼ centimeter a minute, of one-tenth the penetration, would cause the greater number of rejections. The requirements for ductility at 39.2° F., ¼ centimeter a minute, of one-tenth the penetration, the film test, and fixed carbon, would cause the least number of rejections.

The ductility requirement of one-tenth the penetration minus 1 at a speed of ¼ centimeter a minute at 32° F. is also highly restrictive for both grades. The requirement for solubility in 86° B. naphtha would cause the rejection of approximately one-fourth of the 50–60 asphalts and one-fifth of the 85–100 asphalts. The reaction to the Oliensis spot test would reject 14 of the 39 asphalts of the 50–60 grade and 15 of the 40 asphalts of the 85–100 grade.

Of these restrictive specification requirements, as a group, the California asphalts failed to pass the greatest number and the Mexican asphalts, excluding sample 8, failed to pass the least number. The Oliensis spot test indicated that sample 8 had been overheated in the refining process. Those asphalts that show high xylene equivalents generally fail to pass a larger number of these requirements than do those that are homogeneous or only slightly heterogeneous. Next to the Mexican asphalts in the number of failures were the Venezuelan asphalts with negative spots and the Arkansas products. The materials from the other fields showed considerable variation in their ability to pass these specification requirements.

Among the Mexican asphalts of the 50–60 grade, excluding sample 8, there was one sample that failed one requirement, three that failed two requirements, and one that failed three requirements. For the other materials, only sample 18, Venezuelan; sample 22, Arkansas; and sample 37, a Mexican-Texas blend, had as few failures as sample 9, which failed three requirements.

In the 85–100 grade, again excluding sample 8, one Mexican asphalt, sample 7, passed all the special requirements, and four Mexican asphalts, samples 9, 10, 11, and 12 failed to pass one requirement only. One Venezuelan asphalt, sample 18, failed to pass one requirement, and two Arkansas asphalts, samples 21 and 22, failed two requirements.

Undoubtedly the group of materials under consideration includes some asphalts representative of materials that have had unsatisfactory service records. However, there are also included materials from other sources that have had excellent behavior under most severe climatic and traffic conditions. Except for slight deviations, all of these asphalts met the specification requirements that have generally been considered as standard. The failure of such a large number of materials from sources that have been considered satisfactory, as shown in table 17, to pass many of these special requirements gives ample justification for all attempts to discourage the use of discriminatory requirements that are not true measures of quality.

It would seem from a consideration of the known service performance of similar materials and from a study of the test data presented in this report that these special tests are essentially tests that assist in the identification of source or the method of processing or that they are measures of special characteristics. There has been no definite correlation between the test data on asphalts and their performance in service to indicate that any of the tests discussed in this report are true measures of quality or durability. Specifications for asphalt cement, therefore, will remain inadequate until such correlation is provided or more satisfactory tests are developed.

AASHO APPROVES STANDARDS FOR PAVEMENT MARKINGS

The article "Marking and Signing No-Passing Zones on Two- and Three-Lane Roads," published in the December 1939, issue of PUBLIC ROADS, described the systems of marking now used by various States and discussed standards recommended by committees of the American Association of State Highway Officials.

"A Policy on Criteria for Marking and Signing No-Passing Zones on Two- and Three-Lane Roads," and "Standards for Marking and Signing No-Passing Zones," as presented in the article on pages 193 and 202, have recently been approved by the American Association of State Highway Officials.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF FEBRUARY 29, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,256,760	$ 3,022,878	256.0	$ 4,686,781	$ 2,321,828	135.7	$ 1,828,891	$ 907,935	76.2	$ 3,866,067
Arizona	1,904,345	1,350,108	94.4	1,867,529	1,270,126	93.7				1,955,941
Arkansas	5,022,693	3,998,804	226.4	1,271,679	708,282	52.4	1,006,072	497,444	54.3	1,978,950
California	5,158,113	2,783,028	79.1	5,233,953	2,767,418	100.1	2,550,850	1,289,600	40.1	5,196,435
Colorado	3,860,230	2,134,103	89.3	1,587,936	893,102	44.6	12,516	7,054		3,647,315
Connecticut	871,263	431,585	10.0	1,448,790	722,376	15.0	1,120,441	546,721	9.4	1,459,741
Delaware	844,413	403,702	31.3	1,133,702	565,363	7.2				1,587,136
Florida	2,475,449	1,234,877	15.4	4,382,931	2,191,241	98.2	986,848	491,756	51.4	2,586,577
Georgia	4,020,849	1,935,439	222.5	4,922,376	2,461,188	252.3	3,156,303	1,578,152	106.3	7,235,807
Idaho	2,204,320	1,298,965	113.9	874,808	534,106	63.3	226,215	119,244	18.8	2,375,483
Illinois	8,733,950	4,345,913	195.7	5,476,202	2,736,112	89.0	3,554,050	1,777,025	91.3	5,500,757
Indiana	4,039,922	2,003,225	78.6	4,621,687	2,304,437	100.7	2,310,057	1,154,763	45.2	3,625,518
Iowa	3,932,852	1,826,856	190.7	3,638,250	1,616,988	107.9	1,241,130	586,800	54.6	3,048,054
Kansas	3,510,145	1,724,601	175.9	2,014,624	1,006,433	117.4	3,669,350	1,831,155	168.9	6,310,103
Kentucky	3,352,373	1,664,158	112.3	1,755,562	876,225	31.0	1,898,527	949,264	38.3	4,314,738
Louisiana	716,918	355,250	23.9	12,124,062	3,110,897	46.7	1,727,450	846,580	50.7	3,715,371
Maine	2,133,320	1,066,794	52.4	747,557	373,778	16.7	13,240	6,620	.6	1,310,922
Maryland	2,712,006	1,311,116	35.8	1,584,275	783,325	30.9	685,200	297,500	8.1	2,466,054
Massachusetts	3,134,614	1,564,618	25.1	715,245	356,778	5.7	1,569,461	781,488	12.5	3,904,093
Michigan	4,810,287	2,340,742	115.4	2,972,890	1,398,444	100.5	4,286,380	2,143,190	130.0	3,948,679
Minnesota	5,717,758	2,815,774	383.0	3,511,758	1,738,418	162.6	848,521	423,690	45.8	6,218,258
Mississippi	5,504,502	1,873,111	256.5	5,175,058	2,248,645	160.4	2,928,082	1,240,436	187.2	2,786,226
Missouri	3,270,585	1,631,445	140.7	3,365,490	1,655,176	104.2	4,175,165	1,651,907	132.1	6,837,576
Montana	3,291,920	1,861,825	189.8	2,175,159	1,233,267	108.8	1,419,970	804,779	136.3	5,099,011
Nebraska	4,516,205	2,230,935	375.0	3,392,053	1,622,632	324.8	2,615,219	1,236,622	324.0	4,289,201
Nevada	1,113,758	943,594	53.6	1,102,994	947,042	52.6	237,674	204,112	8.7	1,900,975
New Hampshire	825,547	406,051	27.2	711,819	348,783	15.5	73,301	36,650		1,497,489
New Jersey	858,420	420,086	7.1	4,963,808	2,480,954	39.1	1,300,040	650,020	11.1	2,299,140
New Mexico	2,055,830	1,232,304	143.0	1,412,907	880,817	78.1	416,857	260,160	25.3	2,312,387
New York	8,639,476	4,247,817	167.9	12,022,032	5,774,651	168.1	1,359,540	594,770	21.4	5,707,400
North Carolina	5,757,901	2,870,090	353.3	3,953,505	1,978,177	184.2	1,162,580	575,405	79.1	3,355,153
North Dakota	266,609	141,147	41.7	1,307,780	711,478	78.0	3,120,047	1,672,283	300.9	4,575,919
Ohio	5,943,593	2,912,343	84.3	8,216,372	4,084,812	61.8	8,027,351	4,013,540	88.6	6,099,060
Oklahoma	1,963,739	1,038,314	104.0	3,007,856	1,595,897	79.3	2,668,622	1,382,708	100.2	4,997,331
Oregon	2,666,441	1,595,619	108.2	3,165,892	1,701,082	106.0	635,670	371,220	40.7	2,333,075
Pennsylvania	9,655,655	4,773,516	113.7	5,493,574	2,601,802	47.7	4,077,628	1,995,224	45.1	6,637,261
Rhode Island	641,516	319,126	7.8	864,692	431,390	8.9	305,995	152,990	2.6	1,301,406
South Carolina	2,168,021	978,200	81.0	1,574,134	713,909	67.5	932,789	437,998	73.7	3,170,702
South Dakota	3,472,672	1,888,151	328.9	2,778,040	1,586,760	307.5	1,551,660	854,470	237.3	4,292,107
Tennessee	3,725,769	1,778,985	89.3	2,649,508	1,324,754	53.4	2,359,848	1,179,924	45.9	4,906,175
Texas	12,096,592	5,940,219	632.0	9,920,766	4,905,237	484.8	3,046,977	1,507,279	178.4	8,986,552
Utah	2,283,941	1,624,385	98.1	625,885	453,575	46.8	249,525	164,420	4.6	1,828,049
Vermont	732,261	351,856	18.4	711,393	355,506	22.8	122,754	61,355	4.9	861,337
Virginia	2,363,545	1,178,396	76.9	2,499,919	1,194,905	54.8	779,106	389,553	23.5	2,680,211
Washington	2,169,011	1,098,580	38.4	3,382,486	1,771,986	27.1	869,498	458,400	24.8	1,760,367
West Virginia	1,905,237	1,008,304	50.0	1,680,476	836,882	47.7	943,050	467,220	20.3	2,902,027
Wisconsin	5,189,418	2,551,264	187.9	5,021,815	2,464,880	153.2	55,948	25,805	2.4	4,438,708
Wyoming	1,501,346	911,084	141.3	1,263,852	796,581	134.8	765,542	462,676	72.0	1,461,554
District of Columbia	373,200	186,600	2.5	233,724	116,862	2.4	386,736	192,350	2.9	552,313
Hawaii	359,892	175,815	4.3	790,506	381,824	13.1	601,757	299,199	9.8	1,597,423
Puerto Rico	706,072	351,850	14.1	1,306,457	646,785	25.3	65,005	32,220	.2	904,565
TOTALS	171,481,254	88,133,548	6,464.0	161,342,549	78,583,916	4,730.8	79,945,438	39,611,676	3,206.5	178,622,699

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF FEBRUARY 29, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$499,424	$243,701	27.8	$937,932	$367,393	32.9	$633,000	$315,350	34.0	$611,507
Arizona	285,640	204,680	33.4	147,771	106,060	18.3	74,936	17,866	4.7	425,746
Arkansas	928,767	789,307	90.7	103,013	79,812	8.2	91,373	55,810	14.4	347,202
California	821,618	467,084	36.1	389,944	212,124	7.4	273,540	152,732	23.0	1,099,991
Colorado	897,393	473,798	31.3	217,430	99,681	5.9	30,502	4,429	1.3	280,469
Connecticut	172,310	72,417	2.9	100,760	50,197	.2	210,228	99,380	3.1	234,223
Delaware	80,623	39,067	17.5	69,537	34,768	7.8				309,475
Florida	880,547	435,557	31.3	35,029	17,515	.6	101,240	50,620	1.3	563,391
Georgia	343,588	166,234	44.2	156,859	78,430	15.1	447,483	223,742	45.5	1,302,099
Idaho	458,334	248,575	45.1	128,663	78,481	12.8				293,770
Illinois	1,281,443	628,081	78.8	998,500	443,950	56.7	975,250	472,625	39.0	728,405
Indiana	893,825	440,833	77.7	299,900	148,746	18.3	229,992	114,950	18.1	948,527
Iowa	344,063	164,873	128.5	1,098,526	523,360	120.1	1,054,397	497,075	246.4	874,732
Kansas	77,478	38,723	44.4	663,014	331,507	10.9	124,046	62,023	15.0	1,538,013
Kentucky	1,096,826	325,230	71.4	552,881	182,184	37.1	584,600	188,840	55.8	425,368
Louisiana	806,073	377,985	67.2	267,159	133,579	23.7	157,108	78,495	14.5	458,084
Maine	431,791	214,250	25.4	72,824	35,206	3.5	8,300	4,150	.5	138,607
Maryland	281,641	137,157	19.2	72,696	24,348	3.3	45,300	22,650	1.1	474,694
Massachusetts	373,212	185,203	9.2	284,559	140,915	5.9	158,713	78,674	4.2	591,071
Michigan	1,307,572	639,902	113.7	914,039	460,409	45.8	490,325	236,683	29.0	853,435
Minnesota	882,345	435,080	122.0	307,958	153,979	17.6	149,332	42,795	12.5	1,431,189
Mississippi	366,500	183,250	40.7	945,562	460,996	52.1	98,000	48,900	.5	720,893
Missouri	908,174	446,840	136.5	486,413	242,910	41.7	350,008	145,167	48.2	880,260
Montana	835,992	474,167	71.6	224,934	127,101	23.3	460,458	260,410	52.9	774,385
Nebraska	991,190	473,800	205.4	378,287	188,723	47.9	208,743	104,372	40.2	553,652
Nevada	164,206	134,299	25.0	197,591	169,920	25.5				228,342
New Hampshire	63,415	30,804	2.3	118,342	55,625	3.2	21,867	10,676	1.0	195,985
New Jersey	332,425	163,218	12.1	306,190	153,095	12.3	310,690	155,345	6.0	522,893
New Mexico	478,009	292,043	42.1	266,287	166,062	13.8	101,564	63,386	13.1	257,615
New York	1,762,199	865,669	90.9	1,666,975	802,543	45.1	549,766	204,657	12.9	783,234
North Carolina	988,610	492,157	98.5	574,933	290,078	53.6	287,020	135,679	25.4	480,966
North Dakota	115,841	61,370	8.2	62,121	34,895		150,963	80,915	3.8	1,012,290
Ohio	697,494	347,437	41.8	665,090	339,320	25.6	2,305,080	1,101,397	77.0	1,044,378
Oklahoma	260,138	133,815	10.5	534,156	284,214	40.1	340,230	181,028	25.7	1,076,856
Oregon	614,073	347,752	67.6	275,420	128,337	29.9	136,199	81,480	20.2	427,194
Pennsylvania	2,081,304	1,024,235	117.1	1,253,808	620,799	43.1	872,489	435,482	22.8	435,923
Rhode Island	93,827	46,890	2.1	81,236	40,618	.2	148,012	73,980	3.4	95,452
South Carolina	562,159	228,890	56.9	251,420	99,517	22.6	417,800	142,644	29.4	316,531
South Dakota	16,229	8,937	4.1	14,680	9,712					1,280,913
Tennessee	816,719	351,961	31.7	146,416	73,208	.7	132,927	66,463	7.9	1,006,874
Texas	2,367,888	1,166,398	261.9	1,230,763	604,983	134.5	187,720	86,655	35.3	1,399,677
Utah	230,453	126,765	38.5	121,120	76,528	8.3				297,598
Vermont	143,680	69,602	5.6	164,292	50,336	6.3	176,900	52,029	5.8	77,622
Virginia	649,933	315,482	65.5	375,010	163,055	12.8	189,020	92,878	19.1	383,337
Washington	581,930	303,948	43.1	281,527	147,657	16.9	97,856	51,200	4.8	401,162
West Virginia	145,150	72,575	8.3	331,265	165,632	18.0	144,168	72,084	8.0	444,959
Wisconsin	883,153	438,847	34.5	321,053	160,190	4.7	379,319	179,040	4.4	833,634
Wyoming	470,702	286,620	26.0	329,482	195,846	23.4	53,943	34,052	15.5	189,322
District of Columbia	98,700	49,350	1.0	17,492	8,246	.2	24,200	12,100	.3	76,554
Hawaii	89,392	44,391	3.7	376,848	188,928	11.0				159,806
Puerto Rico				224,465	109,130	12.8	105,078	51,475	4.4	109,023
TOTALS	30,953,998	15,709,249	2,671.0	20,042,172	9,860,848	1,181.7	14,089,685	6,642,383	1,051.4	30,397,328

U. S. GOVERNMENT PRINTING OFFICE: 1940

PUBLICATIONS *of the* PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF FEBRUARY 29, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 950,297	$ 946,632	10	2		$ 349,007	$ 348,924	8			$ 513,500	$ 513,500	2		5	$ 884,303
Arizona						518,060	515,813	4								400,746
Arkansas	184,063	184,017	3			672,928	666,423	3		3	325,002	323,920	4	1	6	814,464
California	1,156,170	1,155,003	8		1	1,331,046	1,164,844	8			47,603	47,103	1		3	1,868,824
Colorado	647,329	636,064	5		20	284,341	284,341	1		2	8,562	8,562			4	900,806
Connecticut	26,225	12,665				175,125	169,677		1	7	501,909	496,550	5			581,221
Delaware	7,839	7,839	2								96,433	96,433			16	568,269
Florida	428,094	428,094	2			196,653	192,154	2			23,903	23,903			6	1,438,727
Georgia	200,380	200,380	5		1	534,133	534,133	9	3		467,004	467,004	4	4	20	2,397,613
Idaho	204,578	172,785	3			110,538	110,176	1			234,468	204,238	3			505,509
Illinois	2,607,020	2,547,982	17	4	49	1,875,389	1,743,037	7		26	584,259	469,370	4	1	87	2,487,217
Indiana	760,129	760,129	3	1	58	584,371	584,371	3		47	331,841	331,841	1	2	56	1,384,652
Iowa	821,505	749,256	13		110	439,863	412,800	2		77	439,586	358,766	2		85	1,402,723
Kansas	940,533	937,653	11		8	560,421	560,421	7			535,383	535,383	6	2	11	1,207,964
Kentucky	594,500	589,647	8	4	5	691,827	691,827	6		1	369,025	369,025	5		21	806,127
Louisiana	279,995	279,978	3			674,302	620,806	6			627,885	570,153	11			789,393
Maine	331,672	329,136	3	2		209,733	209,733	2	1		10,004	10,004			4	425,478
Maryland	94,896	85,788	1		3	184,009	184,009	1		2	503,600	430,807	2	1	10	826,925
Massachusetts	400,519	399,288	2	2		122,047	122,014	2			14,320	14,320	1			2,321,561
Michigan	856,209	853,869	6	2	33	831,897	831,897	5		5	663,285	663,285	2	3	13	1,646,878
Minnesota	504,130	478,437	4	4	14	1,433,340	1,432,246	10	2	3	122,930	122,930		1	2	1,679,237
Mississippi	246,300	246,300	6			590,973	590,973	5	1		42,700	42,700	1			1,123,639
Missouri	402,019	400,714	2	1		1,419,571	1,419,571	9	2		110,086	110,086		3		2,270,060
Montana	850,426	850,426	9			217,073	101,991	3			82,912	82,912	2		1	617,711
Nebraska	666,479	665,880	19		21	724,353	724,353	6	2	15	224,562	224,562	1		17	850,005
Nevada	209,333	207,536		4	11	10,353	10,353			2						251,043
New Hampshire	102,433	101,921	7		1	103,328	103,292	3			91,061	91,061	1			379,635
New Jersey	141,570	141,570	1		1	725,086	725,086	2	3		140,550	140,550	1			1,754,825
New Mexico	77,863	77,653	2		1	188,425	188,425	2			138,907	138,907	1		4	614,264
New York	1,832,964	1,798,732	5	7		2,126,073	2,088,413	8	9		1,127,363	1,087,383	4	5		4,021,634
North Carolina	1,196,122	1,161,022	6	5	37	656,388	656,388	8	1	18	341,585	341,585	4	1	12	1,138,911
North Dakota	532,101	483,652	7	1		391,290	391,290	5			119,190	119,190	2		11	810,573
Ohio	541,190	526,190	8	1	2	1,502,813	1,431,861	8	2		998,000	998,000	5	1	3	3,356,519
Oklahoma	305,938	305,402	4		46	277,675	276,775	3	1		707,683	707,683	13		34	2,071,031
Oregon	173,428	171,720	2			134,132	132,837	1			106,751	48,000	2			606,480
Pennsylvania	660,961	656,054	1	2		1,876,028	1,669,036	9	2	1	110,177	110,177	2		1	5,918,697
Rhode Island	446,179	445,450	1	3		7,406	7,406									284,644
South Carolina	483,689	450,307	7	4	7	445,977	434,683	4	4		161,975	161,975	1	2	26	1,072,368
South Dakota	329,447	329,447	3	2	17	79,127	79,127	1	1		59,650	59,650	1		3	1,385,468
Tennessee	283,757	283,757		2	5	590,551	590,551	3	2							1,898,139
Texas	1,953,791	1,918,277	18	3	8	2,195,887	2,132,740	18		11	108,716	96,450	1		8	2,881,380
Utah	290,150	289,996	3		101	102,878	102,878			33	28,300	28,300			10	369,488
Vermont	33,416	28,618			7	206,402	206,402		2							262,583
Virginia	689,809	596,909	7	3	18	215,447	214,577	4			143,827	143,228	1	2	5	1,272,943
Washington	339,534	338,123	3	1	11	193,140	193,140	1	2		91,855	91,855	3		2	743,201
West Virginia	288,981	273,221	5			101,570	101,570	4		1	112,100	112,100	1		3	1,258,032
Wisconsin	883,349	863,369	9		2	858,603	812,506	7	2	9	468,167	439,834	2	1	11	1,377,440
Wyoming	136,007	135,851	1	1	7						85,910	85,910	1			631,367
District of Columbia						359,912	326,368	1								200,203
Hawaii	162,720	160,090	2	1		30,682	30,682	2			236,535	236,525	2		1	261,573
Puerto Rico	49,040	48,840	1			366,272	362,670	8								631,523
TOTALS	26,305,079	25,711,669	248	62	605	28,476,445	27,485,590	212	43	263	12,259,064	11,755,720	105	30	501	65,654,016

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 21, NO. 2

APRIL 1940

RECORDING MOTOR-VEHICLE SPEED AND PLACEMENT

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 2 April 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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NEW TECHNIQUES IN TRAFFIC BEHAVIOR STUDIES

BY THE DIVISIONS OF TESTS AND HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by **E. H. HOLMES**, Highway Engineer-Economist, and **S. E. REYMER**, Assistant Research Engineer

SIGNIFICANT ADVANCES in the technique of studying traffic movement have been made during the past year. New devices have made possible the collection of data needed in studies of highway capacity simply, accurately, and far more cheaply than has heretofore been possible. These devices also have opened entirely new fields of study of the requirements for safe operation of vehicles on our highways.

In 1934, the Public Roads Administration began studies of highway capacity. In these studies, attempts to determine a reasonable or working capacity from theoretical computations were discarded in favor of determinations based upon actual driving practices on representative highways, and an entirely new conception of highway capacity was advanced. This conception differed basically from that of previous approaches to the subject in that the index of highway congestion was based upon the opportunity for the individual driver to travel at his desired speed rather than upon the average speed of all vehicles. Although increasing traffic volumes have been found to reduce the average speed of traffic to some degree, this index, as compared to indexes of the movements of individual vehicles, is relatively insensitive and shows no marked break at which it may be said congestion begins.

Average speeds of traffic may be determined very accurately by an analysis of only a sample of the traffic. Determination of the freedom of movement of individual drivers, however, requires a knowledge of the speed of each vehicle in the traffic stream and its relation to the speed and position of vehicles ahead and behind. Accordingly, methods had to be developed for determining the speeds of all vehicles so that they could be accurately related to one another. This method was described in the February 1939 issue of PUBLIC ROADS.

Studies were conducted under this procedure by the Public Roads Administration in Massachusetts and New York in 1934 and 1935, and in Illinois in 1937 in cooperation with the highway planning survey in that State. However, the scope of the data that could be collected was limited by a variety of factors, and it was practically impossible to observe traffic under a sufficient number of conditions to provide conclusive results.

IMPROVEMENT OF FIELD EQUIPMENT NECESSARY TO OBTAIN ADDITIONAL DATA

The large field force needed was expensive and inconvenient to transport. The necessity of reading license numbers practically confined the work to daylight hours, not only eliminating the study of darkness as a factor in traffic behavior but also preventing observation of some of the heaviest traffic densities. Rain and cold were serious handicaps. The minimum length of section of 0.2 mile precluded observations under physical conditions prevailing for less than that distance. Finally, the office analysis was a tedious and expensive process.

The advantages of the method far outweighed the disadvantages, however. By no previously known method was it possible to obtain the complete information required for these detailed analyses, and the value of the results in showing driver practices under various traffic conditions has been clearly demonstrated. But with the advent of the highway planning surveys and the completion of the basic traffic volume studies, the necessity for more complete knowledge of the factors determining highway capacity became increasingly evident. Traffic density figures for the more heavily traveled roads are of little value unless they can be evaluated in terms of highway capacity. Demand for information similar to that collected in these earlier studies showed that improvement in the field technique was imperative, not to improve the quality of the data, but because more data were needed, added conditions had to be observed, and the results had to be analyzed more quickly.

The requirements of a new method were definitely established on the basis of the earlier experience. The major requirements were as follows:

1. The speeds of all vehicles must be measured.
2. Each speed measurement must be recorded.
3. The time spacings between vehicles must be determined and recorded.
4. The process of determining and recording these items for each vehicle must be completed in less than 1 second.
5. The records must permit segregation of data by traffic lanes, and the direction of travel of each vehicle must be indicated.
6. All measurements and recordings must be entirely automatic, because of the anticipated widespread application of the method.
7. Speeds must be measured with an error of less than 5 percent within the normal range of vehicle speeds.

An extensive survey of existing methods of speed detection revealed that none of the available devices would satisfy all of the requirements and, accordingly, an entirely new mechanical-electrical device was developed. In the conception of this device, the element of recording was perhaps of somewhat greater importance than that of speed determination, for various electronic or other devices would suffice to determine speed reasonably well. Since the Public Roads Administration had obtained satisfactory results with the graphic time recorder in other studies, the new method was developed with that particular type of recording device as its basic element.

The speedmeter is fundamentally a time-measuring instrument consisting of a detecting unit, a timing unit, and a recording unit. The speed of a vehicle is obtained by determining the time it takes in traversing a known distance. The detecting unit functions as two switches placed on the road a definite distance apart. A vehicle in passing over first one and then the other of the two switches starts and stops the timing unit, whereupon the recording unit records the speed of the vehicle. Usually the time required for the entire operation is less than 1 second.

Of the several types of detecting units that may be used, the simplest is of the direct contact type in which each switch consists of a pair of continuous metallic tapes laid across the highway and separated at intervals by some flexible insulating material such as rubber. Detection by lanes may be accomplished by segmenting one of the tapes and using the other as a common return.

Another type of detecting unit, commonly referred to as the "electric eye," utilizes two beams of light projected across the highway and focused on photoelectric cells. This method would seem to be ideal since no visible contacting device is laid across the highway that might attract the motorists' attention. However, this method cannot be easily adapted for use in determining the speeds of vehicles in more than one direction.

Still a third type of detecting unit developed by the Public Roads Administration employs two pneumatic detectors, each consisting of a flexible rubber tubing laid across the road and terminated at both ends by air switches. This unit has been used almost exclusively by the Administration with considerable success, and has proved to be superior to the other types mentioned from the standpoints of operation, portability, ease of installation, durability, and cost. The pneumatic detector has been described in detail in the January 1939 issue of PUBLIC ROADS. Slight modifications of the original design will be discussed later.

NEW TYPE OF SPEEDMETER DEVELOPED

Most of the speedmeters that have been developed by other agencies have employed an electronic timing unit based on the principle of the vacuum tube voltmeter. This consists of a charged condenser biasing the grid of a vacuum tube. When a vehicle actuates one of the switches in the detecting unit, the charged condenser commences to discharge through a resistor. The second switch in the detecting unit, upon being actuated by the vehicle, stops the discharge. The amount of discharge is dependent upon the time consumed by the vehicle in traveling the fixed distance between the two switches of the detecting unit. The remaining charge on the condenser causes a definite measurable current to flow through the vacuum tube. The recording unit is in this case a sensitive meter with a scale which has been calibrated to indicate the speed directly in miles per hour.

The Massachusetts Highway Traffic Research Project developed a speedmeter[1] using a timing unit as described above. A more elaborate electronic speedmeter was developed in Illinois by Reich and Toomin.[2] Careful studies were made of both of these electronic speedmeters and although both possessed many desirable characteristics, they also had certain disadvantages.

Development work by the Public Roads Administration resulted in the production of an entirely new speedmeter incorporating a pneumatic detecting unit, an electromechanical timing unit, and either of two types of recording units.

The pneumatic detecting unit is comprised of two rubber tubes, each 28 feet long and terminated at both ends by air switches. An insulated wire installed inside of each tube serves as the positive return lead from the air switches on the far side of the road to the recording equipment. A single insulated wire laid adjacent to one of the rubber tubes serves as the common negative return lead of the electric circuit. Each tube has a dead section approximately 5 feet long which is centered at the center of the pavement. This arrangement permits detection by lanes since each end of the tube is terminated by an air switch and operates independently. The dead section prevents a vehicle from actuating the two air switches at the two ends of a tube simultaneously should the vehicle straddle the center of the pavement.

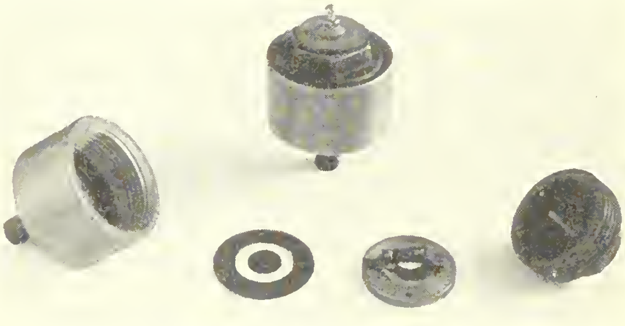

FIGURE 1.—ASSEMBLED AIR SWITCH (TOP) AND COMPONENT PARTS.

Formerly, this dead section was created by simply punching holes through the walls of the tube. This method proved to have some disadvantages when the pavement was wet because the suction resulting from the displacement of air by passing vehicles would draw water into the tube and render the operation erratic. On the other hand, there were certain advantages in perforating the tubing. It permitted equalization of air pressures inside and outside the rubber tubing to be quickly established after passage of the wheels over the tubing; it eliminated the effects of air surges; and it prevented false operation due to changes in temperature. These advantages were particularly important whenever pneumatic detectors were used in connection with traffic counters, but for use with the speedmeter the only significant factor was freedom from temperature effects. In the closed tube that was finally adopted the effects due to changes in temperature were eliminated by making a pinhole in the rubber diaphragm of the air switch. In the final design the dead section was obtained by sealing off a section of the tube.

The air switch employed in the pneumatic detector is shown in figure 1. This unit is much more rugged than the modified engine oil-pressure gage formerly used, and is readily accessible for adjustment. It is constructed of commercially available parts and standard pipe fittings.

TIMING AND RECORDING UNITS DESCRIBED

The timing unit makes use of an electromechanical stepping switch of the type employed in automatic telephony and known as the Strowger 50-point rotary switch. This switch consists of a number of wipers mounted on a shaft which is rotated by a pawl-and-ratchet mechanism actuated by an electromagnet. The wipers are thus stepped around over a number of fixed contacts arranged in an arc. Since the wipers revolve at a definite speed, the amount of their angular movement is a measure of time.

The contacts are grouped in a semicircular rigid unit called a "bank." This bank consists of 4 levels with 25 contacts per level. The wipers have a single wiping end and are arranged in pairs; 1 wiper in a pair is set

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[1] Scientific American, March 1936, vol 154, No. 3, p. 122.

[2] Review of Scientific Instruments, December 1937, vol. 8, No. 12, p. 502.

180° behind the other on a separate level of contacts. The 2 wipers constituting a pair are electrically connected in parallel. Thus, through the 2 wipers, access is had first to 1 level of 25 contacts, and then to another level of 25 contacts, a total of 50 contacts per revolution of the wiper shaft.

The electromagnet driving the switch employs a self-interrupted circuit. Upon operating, it opens its own circuit, then restores, thus closing its own circuit, in principle resembling the action of an electric buzzer. With such a circuit arrangement, the factors which limit the rotary speed are the electrical characteristics of the driving magnet and the mechanical construction of the switch mechanism. The construction of the rotary switch is such that a rotary speed of 55 contacts per second is attained. A complete description of the construction, mechanical operation, and adjustments of the rotary switches has been published elsewhere.[3] It will suffice here to state that the adjustments for the rotary switches are simple and few in number. Operation of the Strowger switch is controlled by a set of quick-acting relays electrically connected to the air switches and actuated by the latter. Motion of the wipers is invariably arrested when one wiper is in contact with one of the 50 contacts on its corresponding level, and through the associated equipment a separate electrical circuit is closed to one of many recording elements in the recording unit.

Under certain conditions a single timing unit can be used successfully on a two-lane highway; however, for this purpose the Administration has adopted a dual timing unit as shown in figure 2, consisting of two Strowger rotary switches adjusted to the same frequency and connected to separate recording units that record speeds independently by lanes.

Either of 2 types of recording units may be employed with the timing unit. One type consists of a standard 20-pen graphic time recorder, mentioned previously, which utilizes a strip chart. By using a novel circuit arrangement made possible by an additional level of contacts on the Strowger switch, the 20-pen recorder was virtually converted into a 39-pen recorder. As shown in figure 3, 2 sets of contacts, consisting of 19 consecutive contacts each, were connected in parallel, each pair of contacts being connected to an individual recording pen. The first set of 19 contacts is connected to pens numbered 1 to 19, inclusive, in the recorder and indicates high speeds in the first half of the speed range. The second set of 19 contacts in conjunction with pen No. 20 corresponds to the second half of the speed range, the low speeds. Pen 20, by virtue of its connection to the additional level of contacts, serves as an indicator of low speeds, differentiating between the high and low speed ranges.

Referring to figure 4, a high speed is indicated by the movement of 1 of the first 19 pens, as exemplified by A, whereas a low speed is indicated by the simultaneous movement of pen 20 and any 1 of the other 19 pens, as shown by B. Pen 1, when operating alone, as shown by C, serves to indicate speeds higher than the calibrated range of the speedmeter. Likewise, pen 20, when operating alone, as shown by D, indicates speeds lower than the normal calibrated range of the speedmeter. Moreover pen 20 serves to indicate the direction of travel of a vehicle opposite to that normally recorded by the recorder, as in the case of one vehicle passing another vehicle within the fixed distance between the detectors.

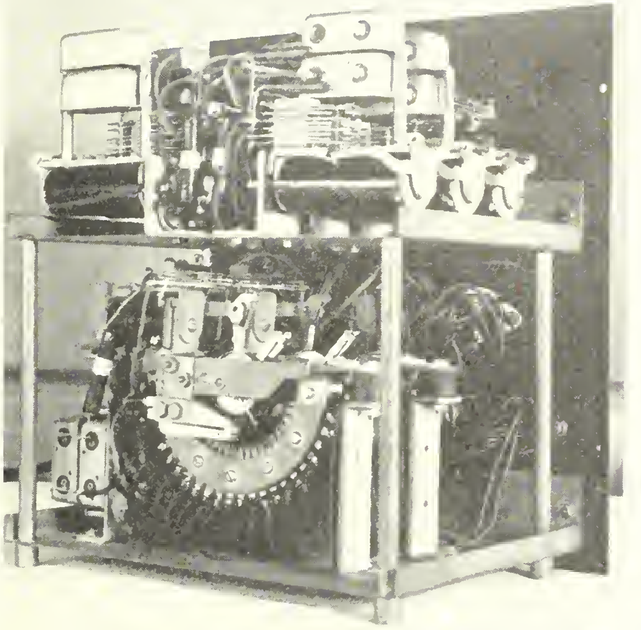

Figure 2.—Timing Unit Used When Studying Traffic in Two Lanes.

To accomplish the recording of the speed and direction of the vehicle performing the passing, the operator depresses a lever-type key that reverses the sequential order of operation of the detectors in the lane occupied by the passing vehicle, upon its approach to the fixed detecting zone. The operator releases the reversing key after the passing vehicle has left the detecting zone. When the reversing key is depressed, pen 20 operates and remains in the operated position for the duration of the recording. A high speed recording is shown by E. It will be noted that in this example, pen 20 was interrupted during the recording of another pen, whereas in the case of a low speed recording shown by F, pen 20 is not interrupted during the recording of another pen. If pen 20 is interrupted during the recording of pen 1, as shown by G, speeds above the range of the speedmeter are indicated. If no other pen records while the reversing key is depressed, pen 20 indicates speeds below the range of the speedmeter as shown by H.

OPERATION OF SPEEDMETER OUTLINED

Figure 5 shows a speedmeter employing two 20-pen graphic recorders. The third graphic recorder shown in this figure is used in connection with a placement detector for recording the transverse positions of vehicles and will be discussed later.

The second type of recording unit consists of a bank of 20 counters of the type employed as telephone call registers. Each counter is individually connected to a separate group of contacts, ranging from 1 to 4 contacts per group, on 2 levels of the Strowger switch, as shown in figure 6. The circuit is arranged so that counter No. 1 records speeds above the calibrated range of the speedmeter and counter No. 20 records speeds below the range of the speedmeter. The calibrated range of speeds is recorded on counters Nos. 2 to 19, inclusive. Each counter records a group of speeds as determined by the number of contacts to which it is connected. In contrast with the other recording unit, this type does not record individual speeds but simply

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[3] Characteristics of Strowger Switches, by K. W. Graybill, Circular No. 1641 Automatic Electric Co., Chicago, Ill.

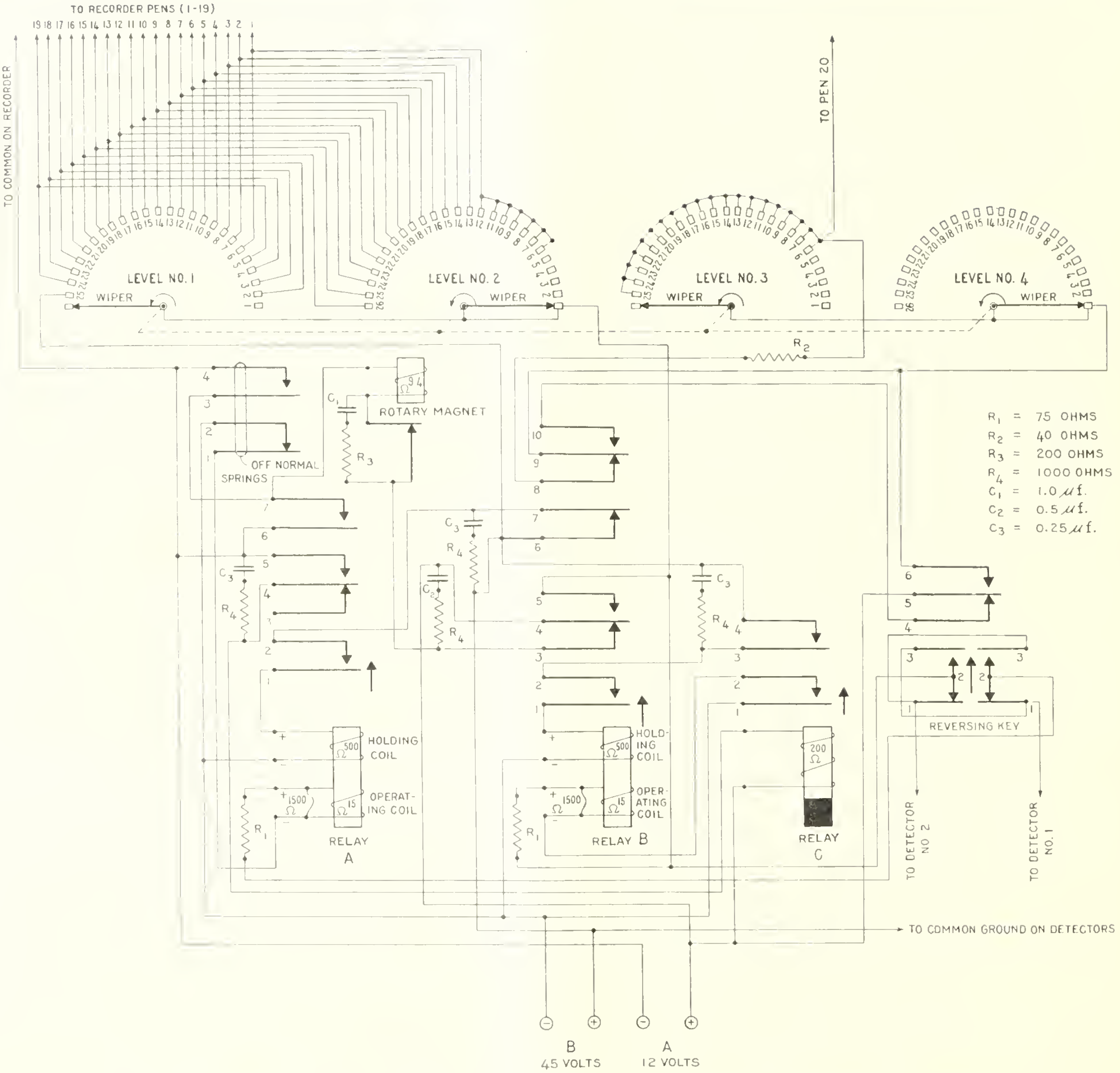

Figure 3.—Wiring Diagram of Single Timing Unit for Use With a 20-Pen Graphic Recorder.

accumulates the group speeds on the series of counters. No provision is made for indicating direction of travel although the circuit arrangement permits the use of the manually controlled direction-reversing key.

Figure 7 shows a speedmeter employing a single 20-counter recording unit.

Referring to the circuit diagram shown in figure 3, the operation of the speedmeter is as follows: Upon actuation of detector 1 by the front wheels of a vehicle, a circuit to relay A is closed causing relay A to pull up in 0.0015 second, closing contacts 1 and 2, 4 and 5, and 6 and 7 simultaneously. Contacts 1 and 2 close a circuit to the holding coil which serves to keep relay A in the operated position. Contacts 4 and 5 close a circuit causing relay C to operate. Contacts 6 and 7 close the circuit to the rotary magnet on the Strowger switch, thus causing all wipers to commence rotating in a counterclockwise direction.

It will be noted that the wipers on levels 2 and 4 sweep over the contacts during the first half of a revolution and wipers on levels 1 and 3 during the second half of a revolution. The upper speed range is covered by the contacts on level 2. The unused contacts on level 2 represent excessive speeds above the desired range. The speed range commences with contacts 6 to 12, inclusive. These are wired in multiple and connected to recording pen 1, which represents speeds above the calibrated range. At the same instant that the wipers commence to rotate, contacts 1 and 2 on the "off-normal springs" open and contacts 3 and 4 close. This is effected by means of the off-normal arm (not shown) mounted on the wiper shaft. Contacts 1 and 2 upon opening serve to open the detector circuit to the operating coil of relay A, preventing false operation by another vehicle during the ensuing period. It will be noted that contacts 3 and 4 on the

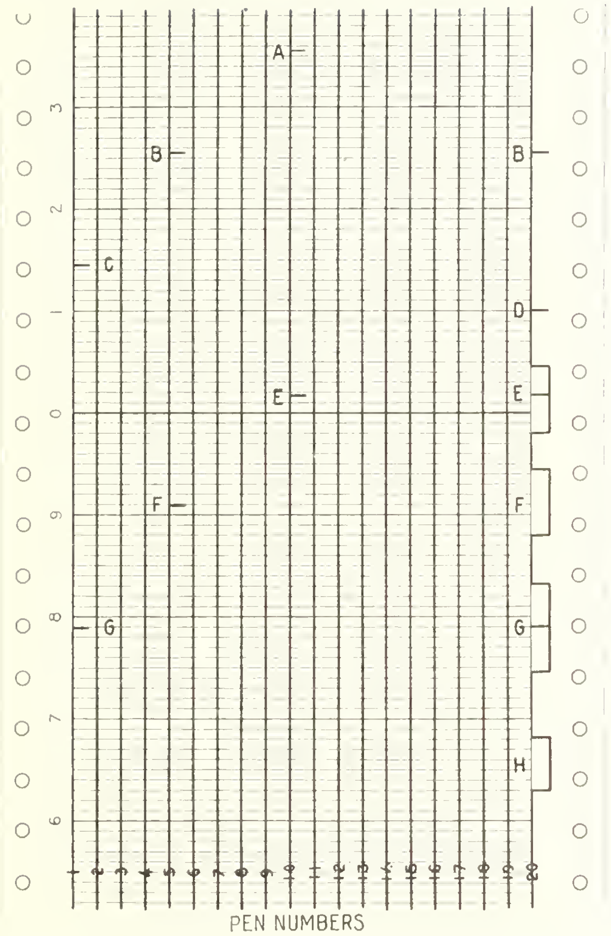

Figure 4.—Typical Strip Chart Record Showing:

	Normal direction	*Opposite direction*
High speed	A	E
Low speed	B	F
Speeds above range	C	G
Speeds below range	D	H

off-normal springs parallel contacts 6 and 7 on relay A. Their purpose will be explained later.

Rotation of the wipers is interrupted whenever a vehicle upon actuating detector 2 causes relay B to operate. Relay B is electrically interlocked with relay C to prevent false operation; furthermore, relay B cannot operate unless relay A has been operated first, since relay A operates relay C. This interlocking circuit through relay C is for the purpose of insuring proper sequence of operation of the detectors and also to prevent false recording of speeds in the interval of time remaining after the speed of one vehicle has been recorded. Relay B upon operating within 0.0015 second simultaneously closes contacts 1 and 2, 4 and 5, 9 and 10, and opens contacts 3 and 4, 6 and 7, and 8 and 9.

The closing of contacts 1 and 2 completes the circuit to the holding coil of relay B causing relay B to stay operated. At the same time, the circuit to the rotary magnet is opened through contacts 3 and 4, thus stopping the rotation of the wipers. Contacts 4 and 5 upon closing complete a circuit to the recording element through the corresponding contact on the level and its wiper. The wiper remains stationary on the level contact for only 0.03 second, which is the time required to actuate the recording element. This time is controlled by the slow-release relay C. The opening of contacts 6 and 7 on relay B causes relay A to drop out to its normal position; whereupon relay A causes the slow-release relay C to open after a delay of 0.03 second. When relay C is de-energized, contacts 3 and 4 on relay C open the circuit of the holding coil on relay B causing relay B to release to its normal position. Relay B upon releasing in 0.0015 second, permits contacts 3 and 4 again to close the circuit to the rotary or driving magnet causing the wipers to resume rotation for the remaining part of the revolution—back to their normal position.

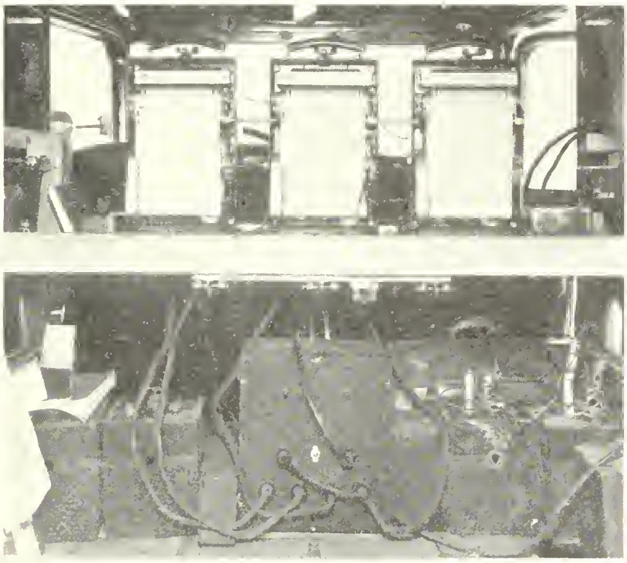

Figure 5.—Graphic Recorders (Top) and Speedmeter Used for Studying Traffic in Two Lanes in Speed-Placement Studies.

The resumption of rotation is referred to as "homing." During this period the detector circuits remain open until the wipers reach their home position, at which time the off-normal arm on the wiper shaft closes contacts 1 and 2 and opens contacts 3 and 4 on the off-normal springs. The closing of contacts 1 and 2 permits relay A to be operated again upon actuation of detector No. 1. The opening of contacts 3 and 4 stops the rotation of the wipers. The speedmeter is again in position for repeating the above cycle of operation.

TWO METHODS AVAILABLE FOR CALIBRATING SPEEDMETER

Either of two methods can be used in calibrating the speedmeter. In the first method, the frequency of the Strowger switch, in contacts per second, is determined from one revolution of the wiper shaft. The off-normal arm, that is mounted on the wiper shaft, opens and closes contacts 1 and 2 on the off-normal springs once for each revolution of the wiper shaft. The time taken for one complete revolution can be determined by means of a photographic oscillograph employing a D'Arsonval galvanometer. The oscillograph is connected in series with contacts 1 and 2 and a battery of low voltage provided with a suitable current limiting resistor to give proper deflection of the galvanometer. Use of contacts 1 and 2 for this purpose is made prior to connecting them into the circuit of relay A, as shown in figure 3.

A typical oscillogram obtained on a high-speed oscillograph using 35-millimeter motion-picture film is

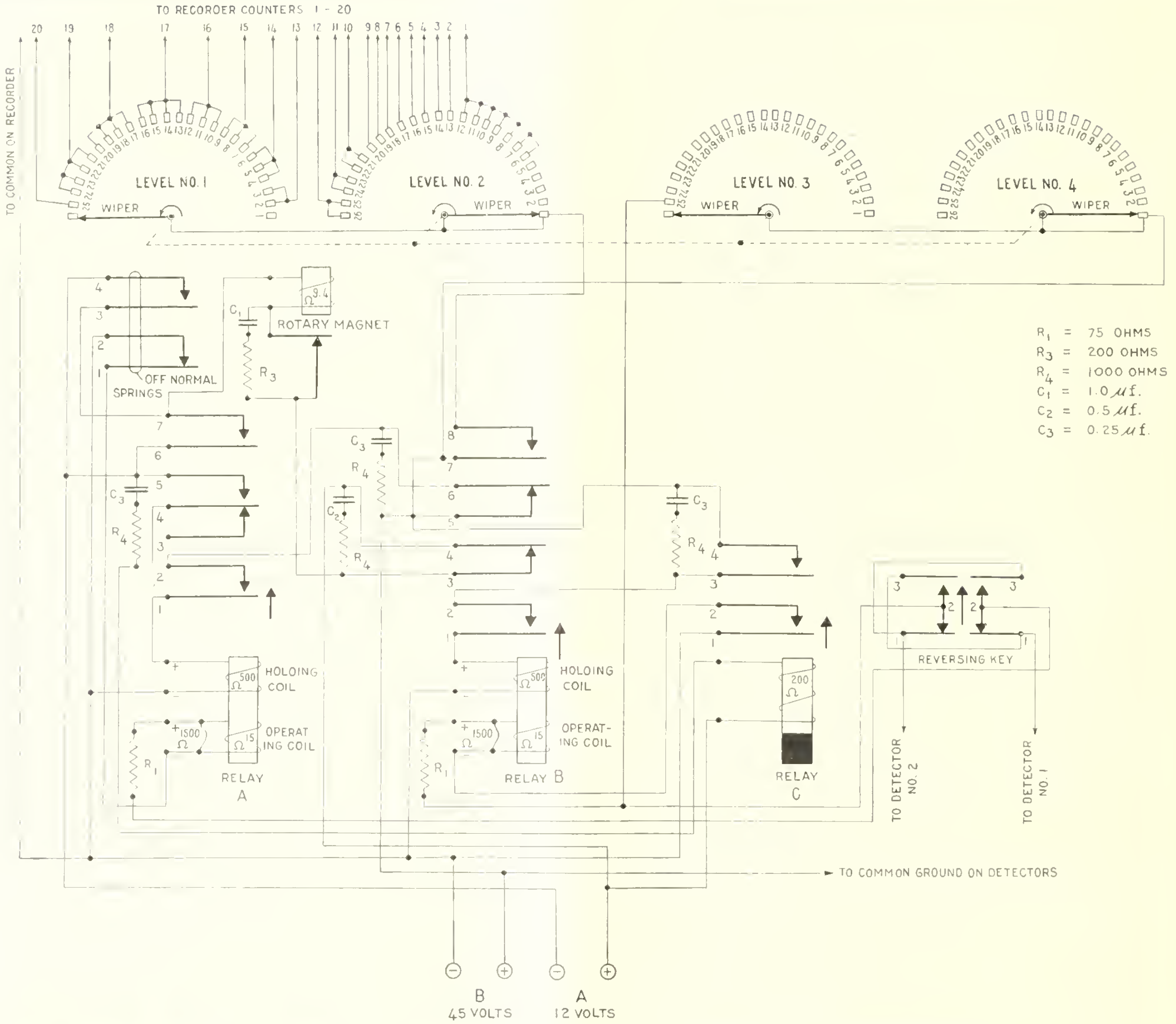

Figure 6.—Wiring Diagram of Timing Unit for Use With a 20-Counter Recorder.

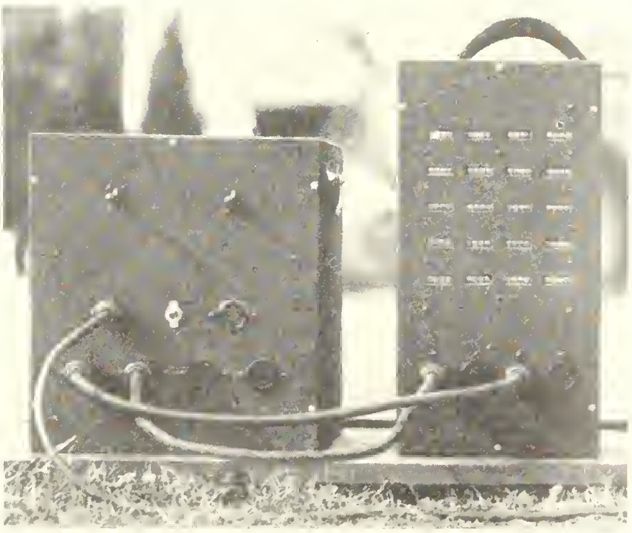

Figure 7.—Speedmeter Using a Single 20-Counter Recording Unit, Used for Studying Traffic in Two Lanes.

shown in figure 8. As determined from this oscillogram, the time required for one complete revolution is 0.860 second. A and B denote, respectively, the opening and closing of contacts 1 and 2 on the off-normal springs. By dividing 50 by 0.860, the Strowger frequency of 58.2 contacts per second is obtained. Having obtained the Strowger frequency, the calibration may be completed graphically from the curves as shown in figure 9, or mathematically as will be shown later.

Referring to figure 9, the Strowger frequency is shown by the straight-line curve. The hyperbolic curves are speed versus time curves for three detector spacings. From these curves the maximum speed for each contact is obtained, beginning with contact 13 which represents pen 2 in the graphic recording unit; contact 14 which represents pen 3, etc. A complete calibration for the speedmeter using a 20-pen graphic recorder and a detector spacing of 24 feet is shown by table 1. A complete calibration for the speedmeter using a single 20-counter recording unit and a detector spacing of 24 feet is shown by table 2.

The second method that may be employed in calibrating the speedmeter uses a stroboscopic photoelectric timing device developed by the Administration. This device, shown in figure 10, consists of an electrically driven turntable whose periphery is precisely graduated into 200 equal divisions. Mounted on the turntable are two vertical shields; one permanently mounted at the zero index, the other adjustable angularly along the graduated scale. Upon rotation of the turntable, these shields serve to interrupt a horizontal beam of light that is projected into a photoelectric cell which in turn actuates a relay. A second relay of the slow-release type is controlled by the photoelectric relay and functions as a single-pole, double-throw switch, switching alternate pulses of the photoelectric relay in response to the interruptions of the light beam. These pulses simulate the actuation of the two detectors by a vehicle and are transferred to the speedmeter through this switching relay which is manually reset for each interval of time used in calibrating the speedmeter.

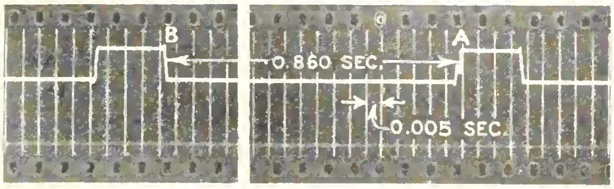

Figure 8.—Oscillogram Obtained for Speedmeter Calibration.

The electrically driven turntable is a modified conventional type using a 110-volt, alternating-current induction motor that is governor-controlled. Since the standard turntables that are equipped with synchronous motors are designed for speeds other than the desired speed of 60 r. p. m., the induction motor type was

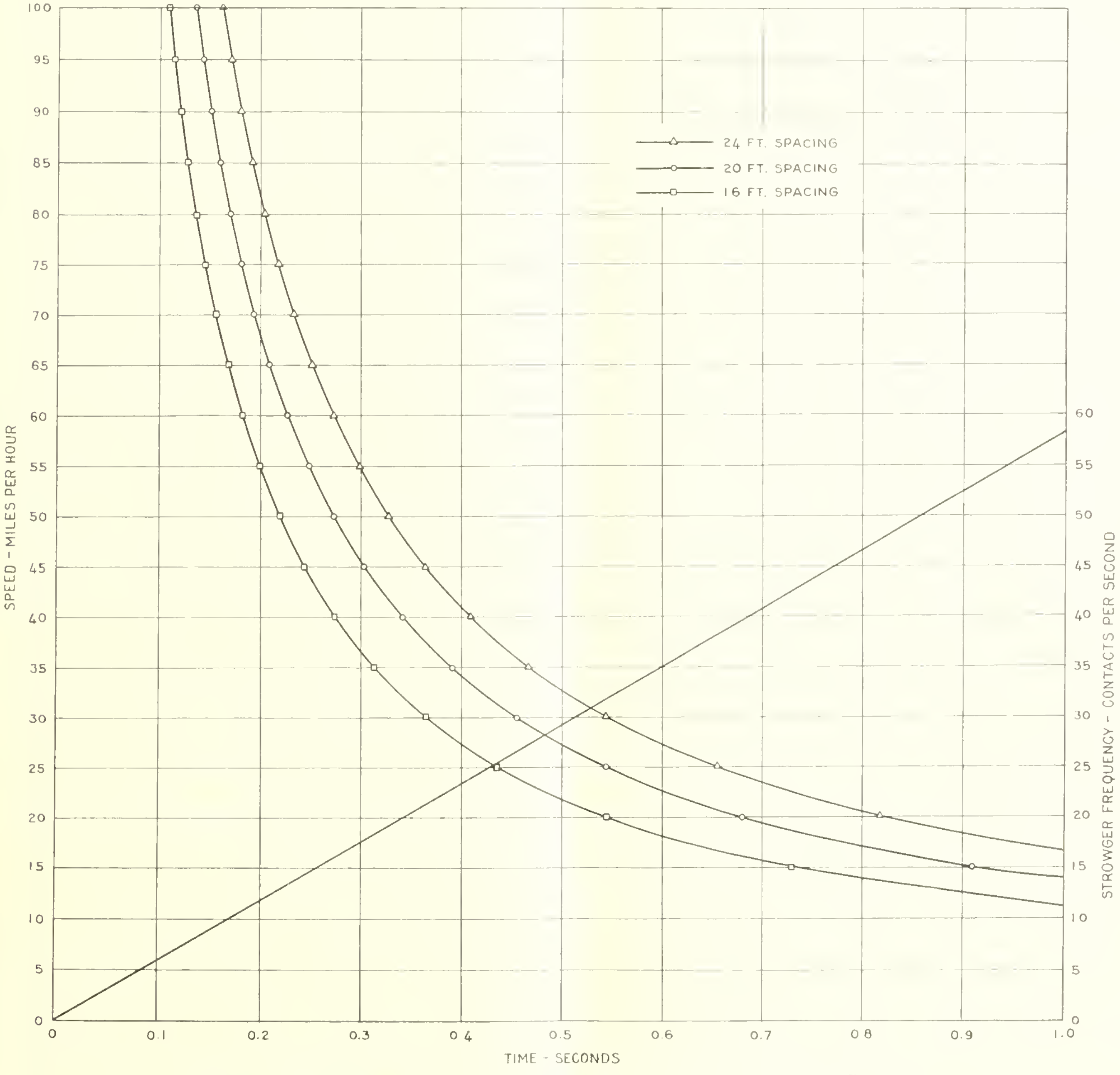

Figure 9.—Speedmeter Calibration Curves for Various Detector Spacings.

TABLE 1.—*Calibration for speedmeter, detectors spaced 24 feet apart*

Contact No. (C)	Recorder pen No.	Maximum speed	Difference	Maximum variation	Average speed
		M. p. h.			*M. p. h.*
12	1	79.3	6.1	±3.05	76.2
13	2	73.2	5.2	2.6	70.6
14	3	68.0	4.5	2.25	65.7
15	4	63.5	4.0	2.0	61.5
16	5	59.5	3.4	1.7	57.8
17	6	56.1	3.2	1.6	54.5
18	7	52.9	2.8	1.4	51.5
19	8	50.1	2.5	1.25	48.8
20	9	47.6	2.2	1.1	46.5
21	10	45.4	2.1	1.05	44.3
22	11	43.3	1.9	.95	42.3
23	12	41.4	1.7	.85	40.5
24	13	39.7	1.6	.8	38.9
25	14	38.1	1.4	.7	37.4
26	15	36.7	1.4	.7	36.0
27	16	35.3	1.3	.65	34.6
28	17	34.0	1.1	.55	33.4
29	18	32.9	1.1	.55	32.3
30	19	31.8	1.1	.55	31.2
31	1 & 20	30.7	.9	.45	30.2
32	2 & 20	29.8	.9	.45	29.3
33	3 & 20	28.9	.9	.45	28.4
34	4 & 20	28.0	.8	.4	27.6
35	5 & 20	27.2	.7	.35	26.8
36	6 & 20	26.5	.7	.35	26.1
37	7 & 20	25.8	.7	.35	25.4
38	8 & 20	25.1	.7	.35	24.7
39	9 & 20	24.4	.6	.3	24.1
40	10 & 20	23.8	.6	.3	23.5
41	11 & 20	23.2	.5	.25	22.9
42	12 & 20	22.7	.5	.25	22.4
43	13 & 20	22.2	.5	.25	21.9
44	14 & 20	21.7	.5	.25	21.4
45	15 & 20	21.2	.5	.25	20.9
46	16 & 20	20.7	.4	.2	20.5
47	17 & 20	20.3	.4	.2	20.1
48	18 & 20	19.9	.4	.2	19.7
49	19 & 20	19.5	.4	.2	19.3
50	20	19.1	-------	------	

TABLE 2.—*Calibration for 20-counter speedmeter, detectors spaced 24 feet apart*

Counter No.	Maximum speed	Difference	Maximum variation	Average speed
	M. p. h.			*M. p. h.*
1	79.3	6.1	±3.05	76.2
2	73.2	5.2	2.6	70.6
3	68.0	4.5	2.25	65.7
4	63.5	4.0	2.0	61.5
5	59.5	3.4	1.7	57.8
6	56.1	3.2	1.6	54.5
7	52.9	2.8	1.4	51.5
8	50.1	2.5	1.25	48.8
9	47.6	2.2	1.1	46.5
10	45.4	4.0	2.0	43.4
11	41.4	3.4	1.7	39.7
12	38.0	2.9	1.45	36.5
13	35.1	2.4	1.2	33.9
14	32.7	3.0	1.5	31.2
15	29.7	2.5	1.25	28.4
16	27.2	2.2	1.1	26.1
17	25.0	2.4	1.2	23.8
18	22.6	2.0	1.0	21.6
19	20.6	1.5	.75	19.8
20	19.1	------	-------	------

selected because it permitted this speed to be obtained by a slight adjustment of the governor that is controlled manually. The speed of the turntable is maintained constant through the use of a stroboscope. A stroboscope disk is mounted on the turntable and a neon lamp with a reference index is mounted directly over the disk. Since the turntable revolves at the rate of 1 revolution per second, each of the 200 divisions on the graduated scale represents 0.005 second. Obviously, any fraction of a second can be obtained by adjustment of the movable shield, although for all practical purposes sufficient precision is obtained in selecting intervals of time to within a half of the smallest division, i. e., 0.0025 second.

In calibrating the speedmeter (fig. 11), several intervals of time are selected and the operation of pens corresponding to these intervals is noted. The maximum time for each interval is established by increasing the time until an adjacent pen in the recorder operates. By plotting the maximum time versus Strowger contacts, a straight-line relation such as that shown in figure 9 is obtained which indicates the Strowger frequency.

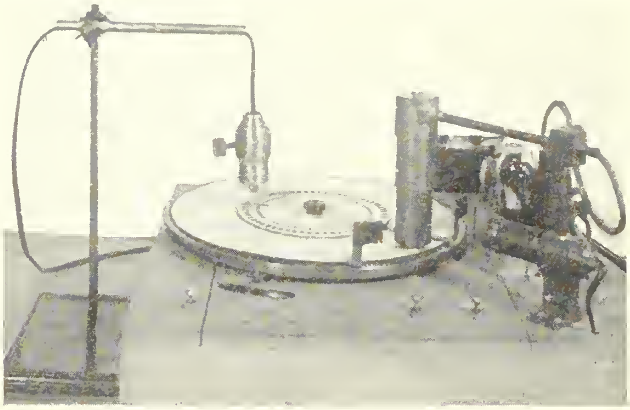

FIGURE 10.—PHOTOELECTRIC STROBOSCOPIC TIMING DEVICE USED IN CALIBRATING SPEEDMETER.

CALIBRATION DATA MAY BE EASILY CALCULATED

The calibration data shown in table 1 may be obtained more rapidly and accurately by a simple mathematical means.

It is assumed that the vehicle is moving at a constant speed during the time of the measurement and it is known that during this time the wiper of the Strowger switch is moving over its contacts at a uniform rate. These two motions can be simply expressed and related for the common time interval so as to give directly the number of the contact point on which the wiper will stop for any vehicle speed and detector spacing.

The motion of the vehicle may be expressed as

$$S=\frac{D}{t}$$

in which S=speed,
D=distance (between detectors),
t=time of transit.

As already pointed out, this relation is shown graphically by the hyperbolic curves in figure 9.

The travel of the Strowger switch wiper may be expressed thus

$$C=ft$$

in which C=the number of the contact on which the wiper will stop,
f=the switch frequency,
t=the time during which the Strowger switch is operating.

This is the equation of the straight line in figure 9.

The time under consideration is common to both movements permitting the equating of the two expressions. This results in the equation

$$S=\frac{Dfk}{C}$$

in which S=vehicle speed in miles per hour,
D=detector spacing in feet,
f=Strowger switch frequency in contacts per second,
C=the number of the switch contact on which the wiper will stop,
k=a conversion constant (0.682) which permits S to be expressed in miles per hour.

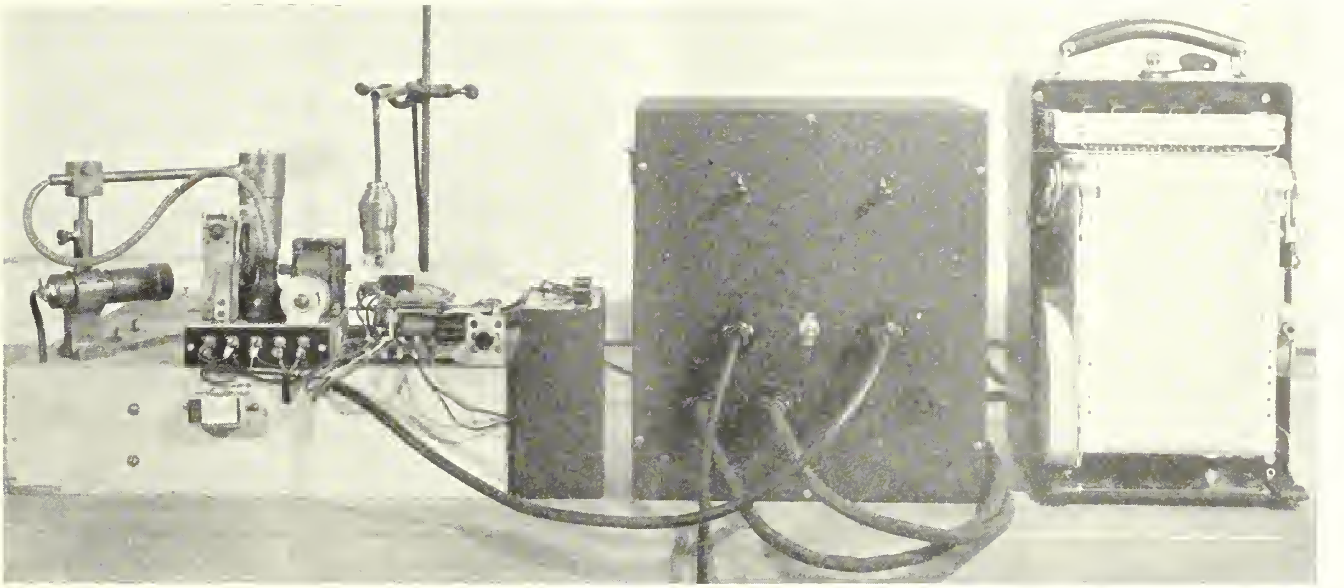

FIGURE 11.—PHOTOELECTRIC STROBOSCOPIC TIMING DEVICE BEING USED TO CALIBRATE SPEEDMETER.

In this expression it is apparent that for any given switch frequency and detector spacing, *S* and *C* are the only variables.

When calculating speed values, such as are shown in table 1, the maximum speeds listed in the third column are based on the corresponding values of *C* given in the first column. The average speed values shown in the sixth column may be calculated directly with the formula by assigning values of 12½, 13½, 14½, etc., to *C*.

Once the speedmeter is calibrated, it is capable of retaining its calibration over a long period of usage. Periodic checks on the calibration have revealed the remarkable stability of the Strowger switch frequency. This feature accounts for its chief advantage in field use, eliminating the necessity of hourly or daily calibration, as required by other types of speedmeters. In one instance, the speedmeter was in use for over 175 hours, recording the speeds of approximately 60,000 vehicles, during which time the calibration remained constant.

The speedmeter can be adapted for other purposes in connection with traffic safety. By certain modifications, it can be used to control the lighting of warning signs to speeders on the highway and simultaneously to actuate a signal at highway patrol police stations indicating presence and direction of travel of the speeder on the highway. The device can also be adapted to control traffic lights in vehicular tunnels for the purpose of regulating the flow of traffic.

TRANSVERSE POSITIONS OF VEHICLES IMPORTANT

Simultaneously with the development of the speedmeter, a device by which the transverse positions of vehicles on the pavement could be automatically recorded was being developed. Such information is needed for two purposes: (1) To determine economical lane widths for various speeds and types of traffic; and (2) to study the behavior of traffic under various highway and roadside conditions. These two purposes are closely related, yet entirely distinct from one another.

Analysis of the transverse positions of vehicles on the highway requires a delimiting of their paths. Under normal conditions, with no interfering traffic, vehicles will travel at such distances from the center or edge of the pavement as the inclinations or habits of the drivers dictate. It is to be expected that the majority of vehicle paths will be concentrated on a portion of the pavement only slightly wider than the average car. If the few extreme cases are excluded, the area along the pavement traversed by the remaining large majority of the vehicles will define the vehicle path on that particular highway. If the cross section is properly designed and vehicular movement is reasonably controlled, it may be expected that this path will lie well within the proper traffic lane. If, however, shoulders are poor or lane widths are too narrow, this path may be shifted toward the center of the pavement until it may even overlap the normal path of vehicles traveling in the opposite direction, with consequent increase in hazard. On a well-designed road, properly used, this path should parallel the center line, neither swerving toward the edge when meeting vehicles moving in the opposite direction nor veering toward the center when passing roadside objects such as culvert headwalls, bridge abutments, posts, or trees.

The determination of the most appropriate combination of lane width and shoulder width and type on normal tangent alinement is thus the first purpose of these studies. The second main purpose is to discover the effect on normal driving practice of curves of varying degrees, of the presence of roadside objects such as guardrails, narrow culverts, and other objects encroaching on the normal shoulder width, and of high curbs, overgrown shoulders, or other features which may have a considerable psychological effect on the driver.

These studies are directed toward economy of highway construction, for they will show, not only how wide traffic lanes should be, but also how narrow they can be, and whether the use of a paved shoulder may have the effect of a greater lane width by inducing vehicles to travel nearer the edge of the pavement. They will also have an equal if not greater value as a means of studying traffic safety, particularly as it is affected by highway design.

Practically all attempts to analyze the causes of accidents have been based upon the questioning of the drivers involved or upon attempted reconstruction of the events immediately preceding accidents. General observations of driver behavior and detailed studies of driving practice, in other phases of the special studies now being conducted by the Public Roads Administration, lead to the belief that it is remarkable, not that so many accidents occur, but that they are so few in number. While many obvious examples of faulty driving are found when accidents occur, a far greater number are found in normal traffic movement in which no accidents occur. The reason that these faulty practices did not result in misfortune is that all the conditions necessary for an accident were not present. If any features of the design of the highway are responsible for or contribute to faulty driving which occasionally produces collisions or mishaps, they can be detected far more readily by a study of the behavior of the tens or hundreds of thousands of vehicles that move without mishap, whether or not safely, than by the most critical examination of the actions of the few that are unfortunate because circumstances combined against them.

The device that has been developed to permit the recording of this vehicle path, or the transverse positions of all vehicles, employs for its detecting element a modification of a so-called positive-contact detector previously developed and patented by the Public Roads Administration and used in other studies. This original detector consisted of two spring-steel strips 20 feet long, separated from one another by means of a rubber arch. This arch is depressed by the passage of a vehicle, thus bringing the strips into contact and completing an electrical circuit. This detecting element was encased in still another arch of similar type to exclude moisture. With lead wires connected to the steel strips, it has served very effectively for the detection of vehicles when used with automatic traffic counters.

Modification of this detector to permit the recording of transverse position required the dividing of one of the steel strips into a number of individual segments, to each of which a separate lead wire is attached. Vehicles passing over the strip then deflect individual sections of the strip and make contacts only at points corresponding to the positions of the wheels on the pavement. Each of the lead wires is connected to an individual pen in the same type of graphic time recorder used in the other phases of this study so that the wheel positions are automatically recorded on the strip chart. Individual segments are 1 foot long, divided into a "live" section 7½ inches long and a "dead" section 4½ inches long. This combination of live and dead sections insures the recording of all wheels but prevents, for most vehicles, the recording of the same wheel on adjacent segments. Vehicles equipped with unusually wide tires or with dual tires will actuate two, or perhaps more, adjacent segments, but it has been found in practice that passenger cars seldom are so recorded. Thus trucks and busses are almost invariably self-identified even without the manual classification normally made as an integral feature of these studies.

EQUIPMENT USEFUL IN MAKING MANY TYPES OF STUDIES

As a result of this development work, there are now available and in operation three distinct types of apparatus adapted either for use independently or in combination with one another. The first is the speed counter with which vehicle speeds may be determined and their records accumulated on a series of counters. This device is useful where general observations of traffic speed are desired. Average speeds and speed distribution may be determined accurately, the results being available almost immediately upon completion of the field work. Typical uses are found in studies of the effect on speed of highway lighting, of enforcement policies, or of various features of the highway design in which identification of particular vehicles is unimportant.

The second type of apparatus is the speedmeter, with which the speed of each vehicle may be measured and, by means of a related graphic recorder, individually and automatically recorded. It is more precise in its record than the speed counter, and is useful wherever the speeds of individual vehicles must be identified. The results may be obtained only after the chart records have been transcribed in the office. The feature of individual vehicle recording is essential in a study of highway capacity.

The third type of apparatus is the placement detector, with which the transverse positions of all vehicles are separately recorded. Since speed is expected to be a factor in vehicle placement, this device is invariably used in combination with the speedmeter.

The speed records collected in conjunction with the placement records are those needed in an analysis of highway capacity. Similarly, if the placement detector is used in conjunction with the speedmeter at the locations where studies of highway capacity are being conducted, information on vehicle paths will be automatically collected as a byproduct of the capacity study. It has been customary, therefore, to collect complete information on both phases of the work at every location studied, regardless of the primary purpose of the operation at any particular spot.

There are distinct differences in the conditions that are important in either of these types of studies, however, even though the analyses are based on the same type of data, and generally it is not possible to schedule a single series of observations to satisfy the requirements of both studies. Accordingly, the work of a particular study is seldom directed equally toward the two different objectives, but rather is concentrated on the one objective of primary importance, with the additional data considered as an inexpensive yet valuable byproduct.

In addition to satisfying these primary requirements, the equipment has made possible a variety of studies not heretofore possible because means of conducting them had been inadequate or completely lacking. Among special purpose studies that have been considered but on which work has not yet been undertaken are the following:

1. *Effect of curvature and superelevation.*—The degree of curve that can be permitted and the superelevation that must be introduced in order to encourage vehicles to negotiate curves without tendency to deviate from their proper traffic lanes may be readily studied with this equipment.

2. *Effect of spiraling.*—By installing the speed and placement detectors at various points along curves and for short distances preceding or following them, the effect of any such treatment may be evaluated. This type of study may be based upon a determination of the average paths of all vehicles at the different points or, by some modification of the apparatus, the paths taken by particular vehicles as they traverse the curve may be observed.

3. *The effect of vehicle length.*—The extent to which long vehicles or combinations such as tractor-truck semitrailers tend to track off line and thus require additional pavement width on curves, whether proper superelevation will eliminate this effect, or whether side slippage of the rear wheels of a trailer, under certain

conditions, might have the reverse effect, can be readily determined. In addition to determining the speeds and positions of such vehicles, the wheelbase lengths may also be recorded without requiring that any vehicles be stopped or otherwise measured.

4. *The effect of special design features, as at intersections.*—The areas of pavement used by vehicles in negotiating intersections or other special highway design features may be investigated by the placement detectors to assist in eliminating unused areas in future designs.

RECORD CHARTS EASILY ANALYZED

In the use of these devices, one speedmeter is normally required for each lane of traffic to be observed, although in very light traffic or for special purposes one speedmeter may be made to serve for more than one lane by providing detectors for each lane and a related switch to make either detector operative as desired. Generally, provision is made for complete records on a two-lane road by incorporating into one unit of equipment two speedmeters and one placement detector each connected to one of three mechanically coupled graphic recorders. For three- or four-lane roads, one unit is installed on either side of the road, and the two are synchronized by simultaneous records made with a telegraph key in series with one pen in each graphic recorder.

Two men are required to operate each unit which is installed in a light panel-body truck or station wagon, enabling work to be conducted under all light and weather conditions. The detectors are placed at the desired point on the road surface and the truck concealed from view well off the road. Figure 12 shows the detectors as installed in a typical set-up. Strips 1 and 4 are the speedmeter detectors, strip 2 is the placement detector, and strip 3 is a detector for an automatic counter used so that the observer may check the rate of traffic flow at any time.

Although the speedmeters function automatically so long as traffic moves normally in the proper lane, the direction of the meters must be reversed in case one vehicle passes another or for some other reason occupies the left lane. Commercial vehicles are classified as they pass, and by means of a telegraph key in series with one graphic recorder pen, each is recorded on the chart in a suitable code symbol in accordance with whether it is a light, medium, or heavy truck or a bus. Tractor-truck semitrailer and truck-trailer combinations can be identified on the chart by the number of axles recorded by the placement detector. In addition to this constant classification of the traffic, the time of day is indicated on the charts at regular intervals, again by telegraph key and a suitable code, so that the chart speed may be accurately checked and the records later broken down into any desired time periods.

Mechanization of the field procedure has virtually eliminated the personal element in the accuracy of the recording, an important factor in work that is rapidly expanding and consequently must be performed by many different individuals. Further, the standardized equipment not only improves the accuracy of the particular studies, but insures that all studies will be made on a comparable basis. Of great importance also, is the reduction in cost. Field work under this new procedure costs less than one-third, and analysis less than one-fourth, the cost using earlier methods.

Two units of this equipment, to permit observations on roads up to four lanes in width, have been in practically continuous use since July 1939 by the Public Roads Administration in cooperation with interested States, generally through the planning survey organizations. Work has been conducted on highway capacity and vehicle behavior in Massachusetts, Illinois, and Texas. Special driver studies have been conducted in Connecticut and South Carolina in conjunction with the American Association of Motor Vehicle Administrators. Special studies of the effect of highway lighting were conducted in Ohio. Further studies are scheduled in other States for the spring and summer of 1940. Other units are being assembled expressly to be loaned to interested States to permit them, with their own highway planning survey personnel, to conduct exhaustive vehicle-behavior studies. One such unit has already been used in Minnesota, and others will be available shortly for further studies already agreed upon. With such active participation long-needed data on the effectiveness of different highway designs will rapidly become available.

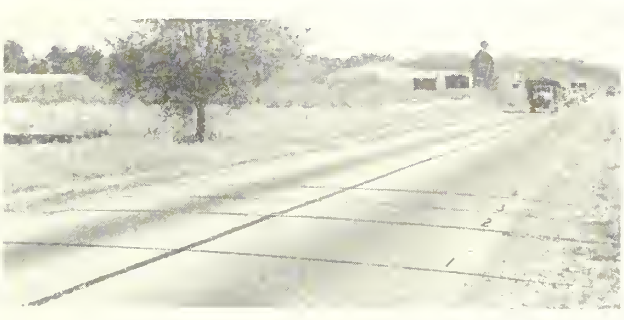

FIGURE 12.—SPEEDMETER AND PLACEMENT DETECTORS INSTALLED ON A TWO-LANE ROAD.

As an example of the type of data obtained, figures compiled from a portion of the records obtained at one location in Massachusetts are presented. This concrete road was 20 feet wide, flanked by 3-foot asphalt shoulders, beyond which were gravel shoulders of variable width. The general alinement of the highway was fairly good and the sight distance at the particular point exceeded a half mile in each direction. Massachusetts has a 30-mile-per-hour prima facie speed limit, but speeds of 50 miles per hour in daylight and 45 at night are generally not considered unreasonable. The figures analyzed here are based on observation of approximately 1,100 vehicles, a number ample for purposes such as determining average speed or speed distribution, but insufficient to permit a breakdown into many subdivisions. The results must be considered, therefore, as illustrative of procedure only and without significance as to the effect of particular highway conditions. Significant results will be obtained only from a critical analysis of similar records covering longer periods at many locations.

Portions of the three synchronized charts on which the records from this section appear are shown in figure 13. The center chart shows the transverse positions. The lines numbered from 1 to 10 represent the south side of the road, on which eastbound traffic normally moved, and pens 11 to 20 represent the north side, or the lane of westbound traffic. The charts above and below are the speedmeter charts, the upper one corresponding to placement pens 11 to 20 and the lower one to placement pens 1 to 10.

The type of vehicle is manually recorded by means of a telegraph key in series with pen 20 of the placement recorder. If it is evident that pen 20 may be actuated by the vehicle in question, the manual record of the classification is delayed momentarily, but otherwise it

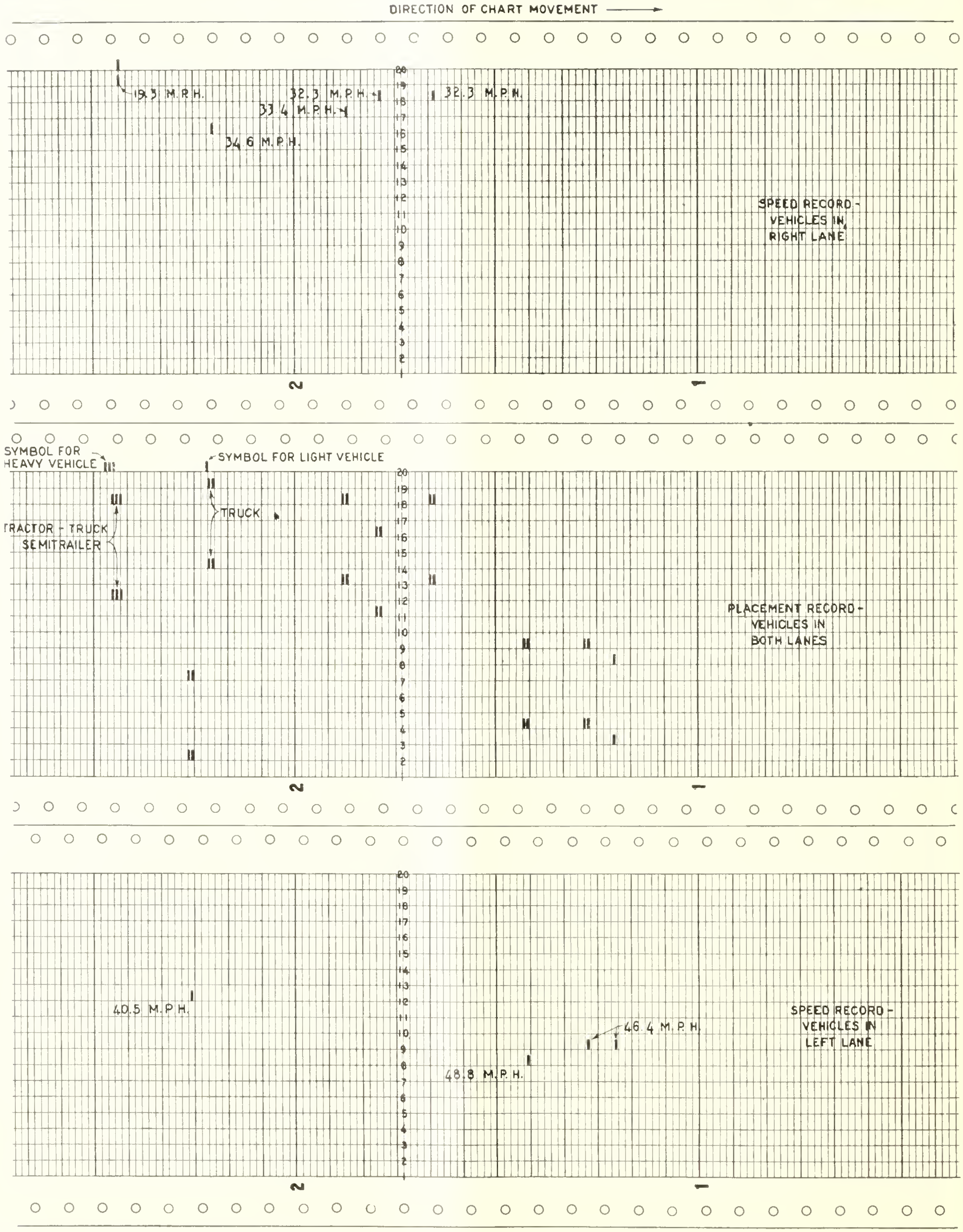

Figure 13.—Portions of Three Synchronized Charts, Showing Samples of Record Obtained on Vehicle Speed and Placement.

appears in line with the wheel records so there is no difficulty in interpretation of the chart records. The longitudinal spacing as well as the transverse position is normally determined from the placement chart, each transverse line representing 1 second in time at the normal chart speed of 6 inches per minute.

The speed of each vehicle can readily be obtained from the upper and lower charts, the mark in each case being in line with the record of the transverse position as shown on the middle chart. The speeds shown beside each mark represent the average of the range of speeds included in the particular speed group. The speed recorded as 34.6 miles per hour, for example, may actually be anywhere between 34 and 35.3 miles per hour.

PLACEMENT DETECTOR GIVES PRECISE RESULTS

Thus for individual vehicles the speeds and speed differences may be slightly in error, but such errors will be compensated for and their effects nullified by the large number of vehicles that will be included in the various speed and speed difference groups.

As mentioned before, the data may be analyzed for purposes of studying both highway capacity and vehicle behavior. Here the primary purpose was to supply data on highway capacity, and the effect of the particular conditions on traffic behavior was secondary. During the hours in which these records were collected, very little change in volume occurred, so no attempt has been made to show here the effect of volume changes on any of the congestion indexes. These indexes have been calculated, however, for later use in studying the effect of various traffic volumes on this section from the complete records.

It was found that the average speed of all vehicles, with the average volume of 375 vehicles per hour, was 41.0 miles per hour; that the average free speed of passenger cars (when no vehicle moving in the same direction was closer than 6 seconds ahead and no vehicle moving in the opposite was closer than 5 seconds ahead or 2½ seconds behind) was 41.6 miles per hour; and the average free speed of trucks was 34.5 miles per hour. The average speed difference was 6.1 miles per hour. It is of interest that these figures all lie within the range for these same indices found on roads of similar alinements in the earlier studies under the method previously used.[4] The close agreement between the average free speed and the average speed of all vehicles, and the relatively high average speed difference show this road to be practically uncongested at this traffic volume.

A further break-down of these vehicle speeds is shown in figure 14. No trucks traveling freely moved faster than 50 miles per hour, and no free-moving passenger car moved faster than 60 miles per hour. The modal averages of these classes of vehicles fell in the 35–39 and 40–44 mile-per-hour groups, respectively. It was of interest that 0.1 percent of all vehicles, traveled more than 70 miles per hour; and that while but 1 percent traveled at 60 miles per hour or faster, all such vehicles were so placed with respect to the other units of the traffic stream that their behavior may have been influenced by the presence of these other vehicles. If so, the influence apparently was not a retarding one. Because the limits of the speed groups as recorded by the speedmeter are not exactly uniform, the speed distributions were first plotted in a cumulative curve from which the number or percentage of vehicles in uniform groups of the desired size were scaled.

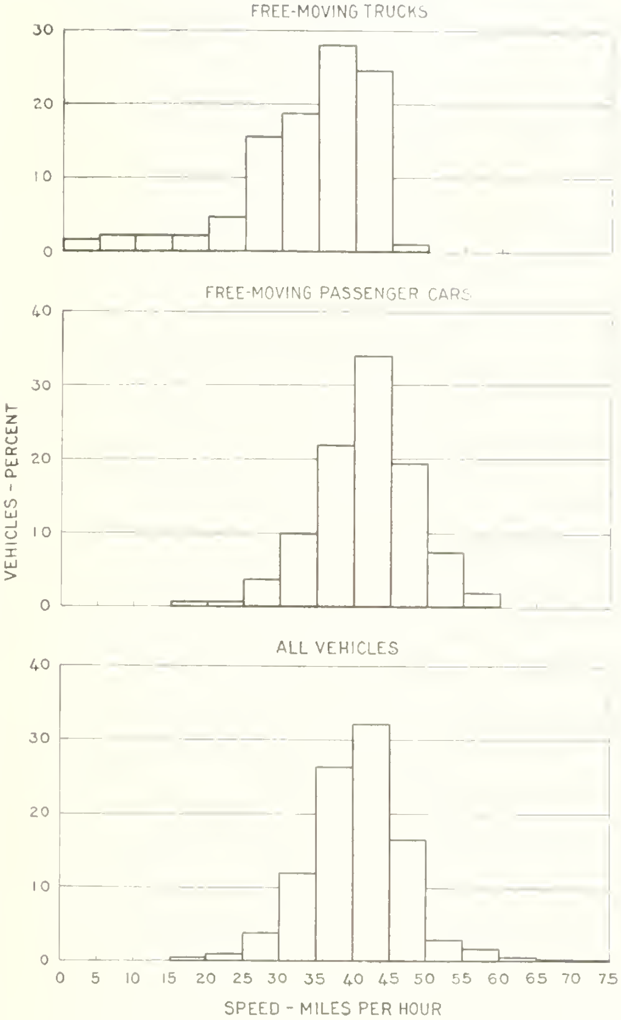

FIGURE 14.—FREQUENCY DISTRIBUTION OF VEHICLE SPEEDS.

The distribution of the time spacings is another index regularly determined in these analyses. These figures are of particular value in coordinating the results of the congestion studies with those obtained in the passing-practice studies. The time spacings of all vehicles traveling in the same direction are plotted as a frequency distribution in figure 15, a further break-down for those below 10 seconds being shown separately.

It is of interest to compare these figures with the general trends of traffic flow discovered in the earlier studies. Almost invariably it was found that between two-thirds and three-fourths of the vehicles traveled at or less than the average spacing. Here 66 percent are found to be traveling at or less than the average of 19.1 seconds. Similarly, previous analyses have shown that approximately half the vehicles travel at half the average spacing or less, and this distribution shows 48 percent not exceeding 9.5 seconds. It was not surprising to find verification of these tendencies. Such close agreement not only confirms the validity of the figures collected under the previous manual methods of recording, but also shows that figures developed from future studies may be combined with those collected in the past work without regard to the method by which any group was obtained. The radical change in the field procedure does

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[4] Preliminary Results of Highway Capacity Studies, PUBLIC ROADS, February 1939.

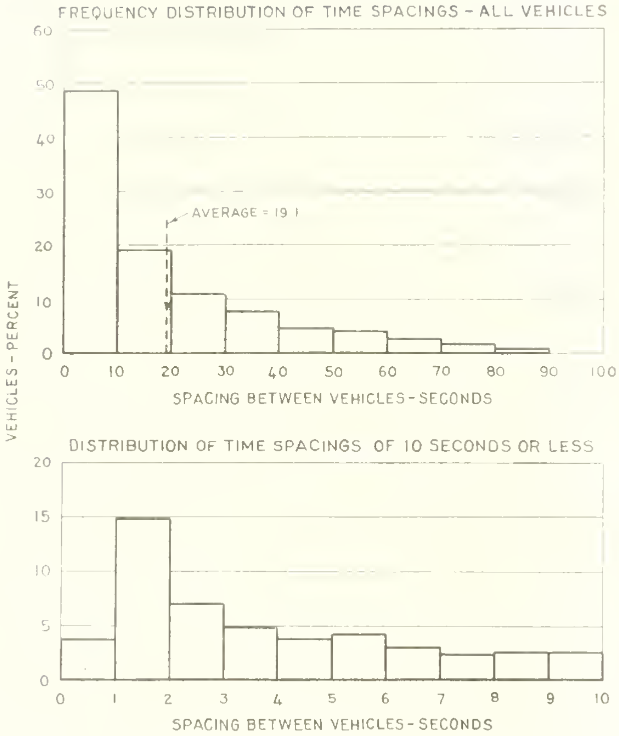

FIGURE 15.—FREQUENCY DISTRIBUTION OF TIME SPACINGS OF VEHICLES.

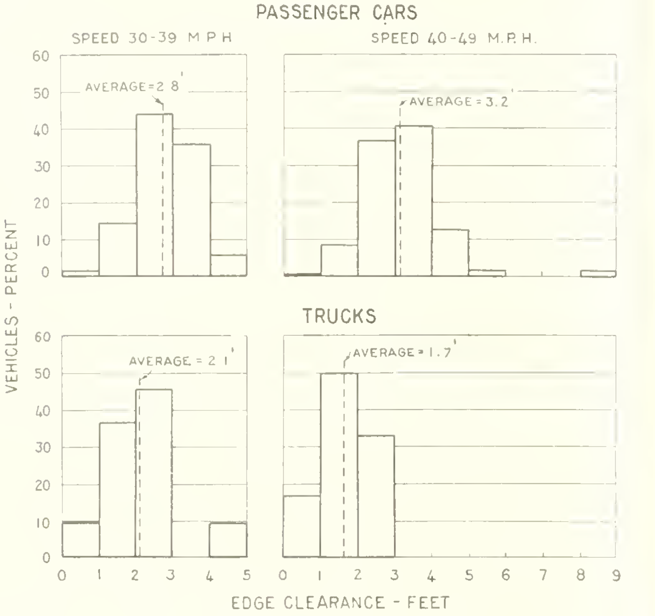

FIGURE 16.—EDGE CLEARANCES OF PASSENGER CARS AND TRUCKS MOVING FREELY.

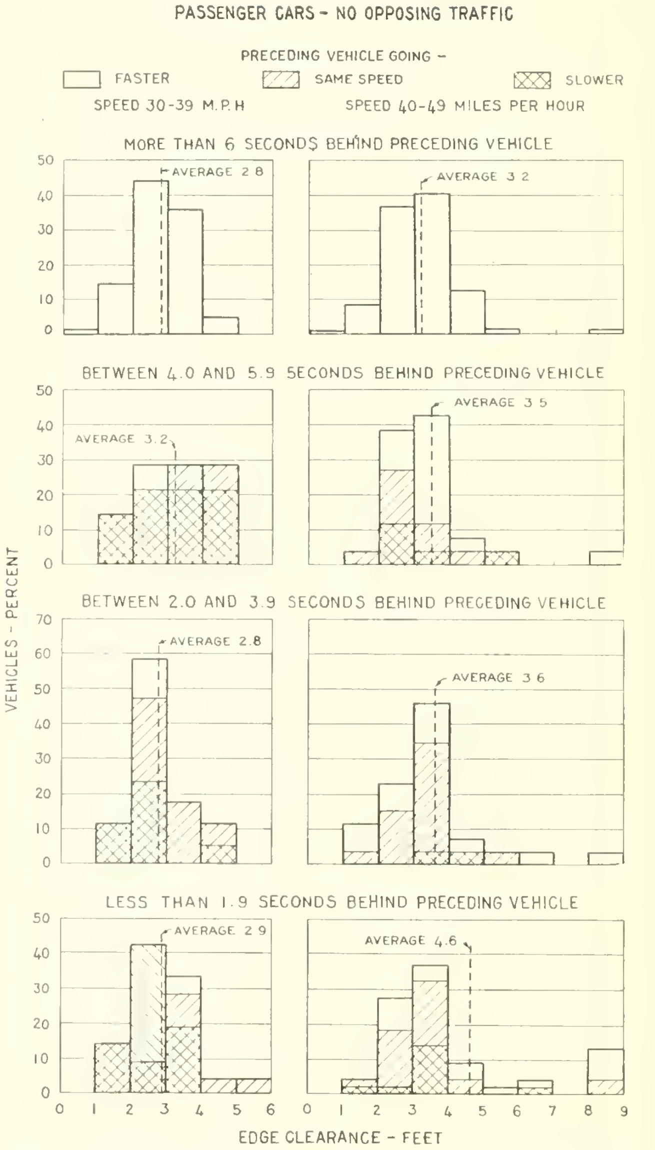

FIGURE 17.—EDGE CLEARANCES OF VEHICLES TRAVELING IN TWO SPEED RANGES AND TRAVELING AT VARIOUS TIME SPACINGS BEHIND PRECEDING VEHICLES.

not, as a consequence, require any change in the treatment of the resulting data.

Many valuable uses will be found for the information on vehicle placement (fig. 16–20). Some of these uses will be shown by the results presented here, but as mentioned before, the significance is severely limited by the size of the sample. First, however, it is of interest to examine the precision of the records. The placement strip will record the position of a wheel of a particular vehicle to the nearest foot only, although if each placement is assumed to be in the center of the segment of the strip actuated, the maximum error in the record of the placement of a particular wheel will be 6 inches. The position of the average vehicle, however, may be found with a high degree of precision. Since the positions of both right and left wheels are recorded, the difference between the positions of the average left and right wheels should equal the tread width of the average vehicle.

In transcribing the field records it is customary, where the record for one side of the vehicle appears on two adjacent segments, to record the wider of the two rather than the average. The field records, therefore, should agree more closely with the rear or wider tread than with the average. The average tread width of all passenger cars was found from the field records to be 4.92 feet. The average rear-tread width of vehicles, as obtained from manufacturers' specifications and weighted approximately in accordance with the occurrence of cars of various makes in the traffic stream, was also found to be 4.92 feet. This absolute agreement between the two is perhaps a coincidence, but it demonstrates that use of the placement strip with 1-foot segments insures satisfactory precision.

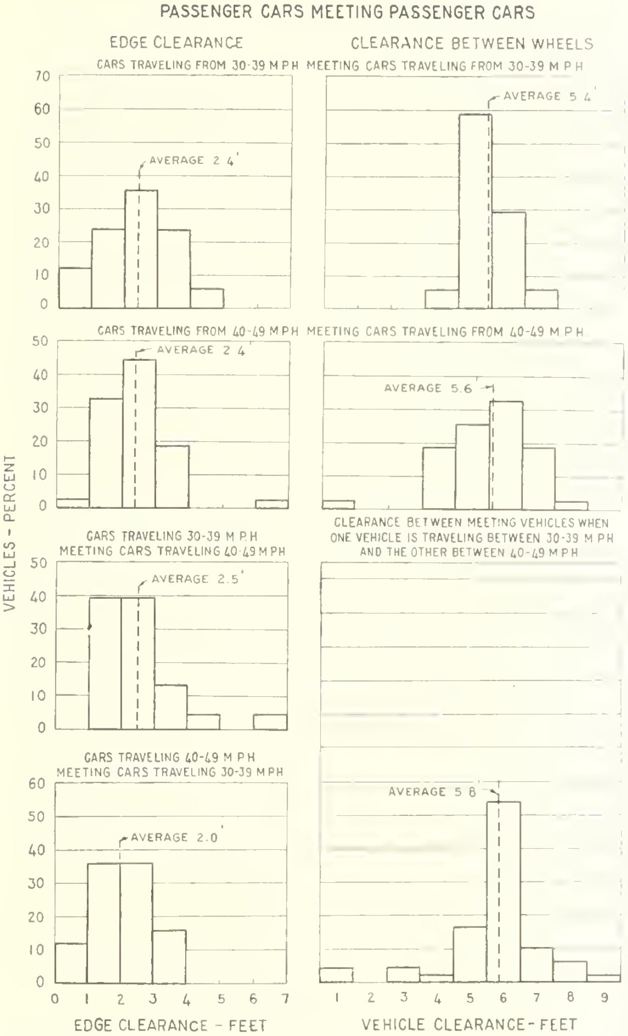

Figure 18.—Edge Clearances and Vehicle Clearances of Passenger Cars Traveling at Various Speeds Meeting Other Passenger Cars Traveling at Various Speeds.

FASTER CARS TRAVEL NEARER CENTER OF ROAD

The transverse position of a vehicle on the road surface is likely to be influenced by a variety of factors both internal and external. The internal factors are largely dependent on the psychological make-up of the driver, and generally are measurable only to the extent that they are reflected in the speed and placement of the vehicle. These psychological factors may be more positively evaluated by subsequent questioning of the drivers in a manner now being explored in cooperative studies with a number of motor-vehicle departments. The external factors may be roughly divided into three broad groups: The physical features at or near the point of study; the presence of vehicles moving in the same direction as the vehicle under consideration; and the presence of vehicles moving in the opposite direction.

Invariably at least two of these factors exert their influence in combination, and the effect of a single factor can never be directly determined. The effect of physical features may be determined with the least complication by selecting for analysis only vehicles that are uninfluenced by other vehicles, although the combined effect of physical features and opposing traffic must also be investigated. In order to include in this category a sufficient number from this particular sample of 1,100 vehicles, free-moving vehicles were considered to be those that had not met another car for 2½ seconds, would not meet another for 5 seconds, and followed a vehicle in the same direction by at least 6 seconds. A vehicle to be met 5 seconds later would be recorded as crossing the placement strip 10 seconds after the car in question, since the meeting would take place midway (in time) between the original positions. Previous studies have shown that drivers may be affected by the presence of a vehicle ahead moving in the same direction when the spacing between them becomes less than 9 seconds. This spacing would be preferable to the 6 seconds used here but would have unduly restricted the size of the sample.

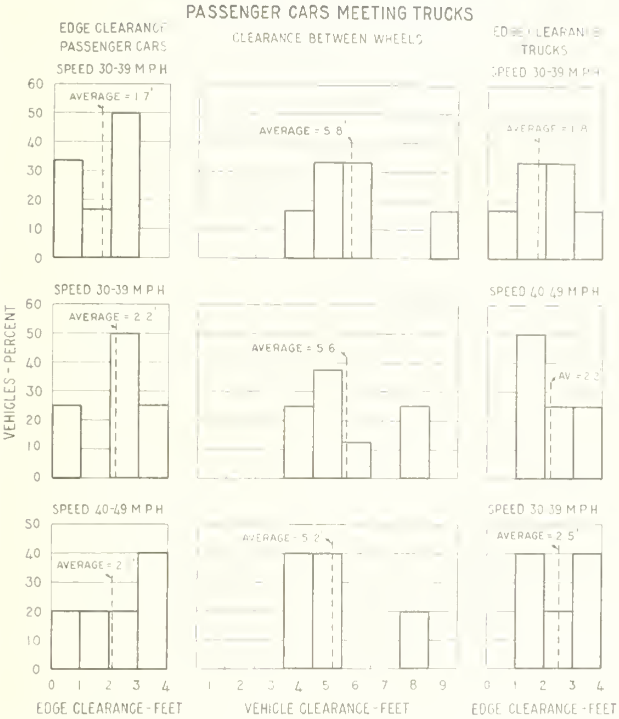

Figure 19.—Edge Clearances and Vehicle Clearances of Passenger Cars Traveling at Various Speeds Meeting Trucks Traveling at Various Speeds.

Figure 16 shows the edge clearances of passenger cars and trucks under these conditions. The edge clearance is the distance between the edge of the pavement and the center of the outside tire. Only two speed groups are shown because in other groups the samples were too small. Under the conditions at that particular point, trucks moved closer to the edge of the pavement than did passenger cars, and the faster passenger cars tended to move somewhat nearer the center of the road than did the slower moving cars. Assuming 6½ feet as the width of a passenger car, 98 percent of the passenger cars traveling between 30 and 39 and 94 percent of those traveling between 40 and 49 miles per hour, were entirely in their own lane, and presumably, therefore, would not have collided with properly driven cars in the other direction even without swerving to the right on meeting these other vehicles. Similarly 95 percent of the slower trucks and all of the faster trucks were entirely in their own lane. Some such index as this will permit comparisons between various cross-section designs on a mathematical basis, so that the effectiveness of any combination of roadway features may be exactly evaluated.

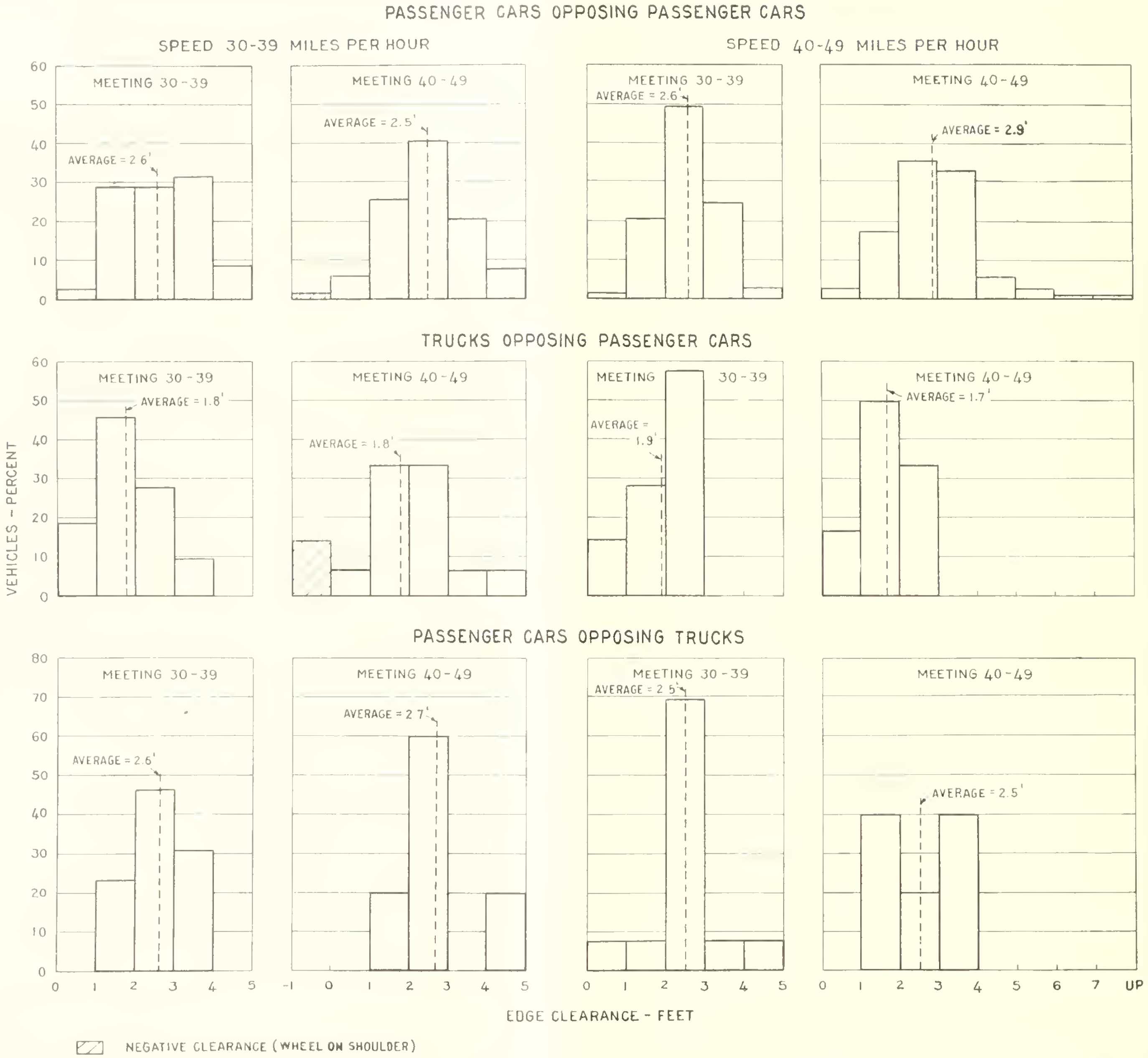

Figure 20.—Edge Clearances of Passenger Cars and Trucks Traveling at Various Speeds Meeting Other Passenger Cars and Trucks Traveling at Various Speeds.

The presence of vehicles moving just ahead of the one under consideration and in the same direction affects vehicle placement, as shown in figure 17. At the top, for comparative purposes, is shown the transverse distribution of vehicles moving freely (with no vehicle closer than 6 seconds ahead). It is of interest that as the distance between vehicles becomes less, the following vehicles tend to travel nearer the center, possibly to provide better visibility of the road ahead. Following of this tendency by successive vehicles of a group would result in the rearmost vehicles assuming highly hazardous positions.

VEHICLES TRAVEL CLOSER TO EDGE WHEN MEETING ONCOMING TRAFFIC

Figure 18 shows the edge clearances and the distances between passenger cars meeting other passenger cars approximately on the placement strip, and figure 19 shows the same information for passenger cars meeting trucks. In this category are included vehicles crossing the strip within 1 second of one another, which is equivalent to meeting or having met within ½ second. In analysis this number was divided into three groups: Those meeting exactly at the strip; those crossing the strip within ½ second; and those crossing the strip between ½ and 1 second of one another. So little difference was found in the averages for the three classifications that they were grouped together.

This figure shows that the average passenger car on meeting another vehicle moves toward the edge of the pavement somewhat, with the faster vehicles traveling as close to the edge or closer than the slower group. This latter fact is of interest because, as shown by figure 16, the average car of the faster group drove normally at 3.2 feet from the edge as compared to 2.8 feet for the average car in the slower group. Drivers of the faster cars allowed an equal or greater margin of clearance on meeting other cars than did slower drivers, but were required to move laterally a greater distance to do so.

A summation of vehicle clearances aids in obtaining a further indication of the accuracy with which the position of the average vehicle may be determined. The two edge clearances, the tread widths, and the clearance between these meeting vehicles should total the pavement width. In the three classifications shown for passenger cars in figure 18, these distances add to 20.0, 20.2, and 20.1 feet, respectively, the maximum error being 1 percent. Such close agreement is neither expected nor attained in the case of the trucks, however, because of the small sample and the greater variation in truck tread widths.

Since the average passenger car is driven somewhat closer to the center of the road when not opposed by other vehicles, and moves toward the edge on meeting another car, it is of interest to find at what distance from the opposing vehicle this lateral movement is begun. If, for example, the effect of an oncoming vehicle becomes noticeable when that car is 10 seconds away, it is reasonable to assume that the sight distance provided should be the distance traveled in 10 seconds by a vehicle moving at the design speed, in order to permit drivers to perform this lateral transition normally. This distance, which might be termed the "meeting" sight distance can, under certain conditions of alinement and highway usage, be more important from a standpoint of safety than mere stopping sight distance.

Measurement of the lateral movement of a vehicle as it meets another is simply a matter of determining the average transverse positions of vehicles when opposing vehicles are various distances away. Since particular distances between meeting vehicles exist only instantly, rather than continuously as for cars moving at the same speed in the same direction, a very large volume of traffic must be observed to obtain a usable sample of "opposing" spacings. From this particular sample it was impossible to relate the transverse positions to opposing spacings closely enough to determine when the lateral transition began. Figure 20 shows, however, the average positions of vehicles possibly affected by opposing vehicles (i. e., having met another vehicle within 2½ seconds or about to meet one within 5 seconds but not meeting another vehicle within ½ second of crossing the placement strip). As should be expected, the average transverse position of all vehicles about to meet others falls between that for vehicles actually meeting others and vehicles moving freely. It is of interest, however, from a point of view of reliability of the data, that this condition was found invariably in all the classifications of speed and vehicle type.

As stated before, the relationships shown here are intended merely to show the manner in which the transverse placement data may be used. With a greater volume of information, additional relationships may be found, and the analyses greatly refined. With such indexes, established for the more common combinations of standards of surface and shoulder design, and horizontal and vertical alinement, the effectiveness of design features need not be based on opinion and judgment; their actual merit may be definitely evaluated.

FIVE BULLETINS ON BRIDGE FLOOR DESIGN AVAILABLE

Five bulletins on bridge floor design, whose availability for free distribution has previously been announced in PUBLIC ROADS, may still be obtained from the Public Roads Administration. The publications, all University of Illinois Bulletins, are:

No. 303—Solutions for Certain Rectangular Slabs Continuous Over Flexible Supports.
No. 304—A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams.
No. 313—Tests of Plaster-Model Slabs Subjected to Concentrated Loads.
No. 314—Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.
No. 315—Moments in Simple Span Bridge Slabs with Stiffened Edges.

The bulletins are the result of a cooperative investigation by the Engineering Experiment Station of the University of Illinois, the Illinois Division of Highways, and the Public Roads Administration of the Federal Works Agency.

The results of this investigation have direct application to practically all modern highway bridges and to many other structural design problems.

A limited number of these bulletins are still available for free distribution. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MARCH 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,590,205	$ 3,189,278	268.7	$ 5,067,117	$ 2,508,413	154.5	$ 1,115,110	$ 554,950	44.8	$ 3,866,067
Arizona	2,042,051	1,447,104	100.1	1,905,859	1,298,649	96.6	55,221	33,200	1.7	1,797,220
Arkansas	5,026,310	3,996,071	226.4	1,616,156	880,291	66.0	1,140,839	564,315	63.3	1,742,802
California	5,552,016	2,977,652	82.1	6,717,815	3,536,324	113.0	1,490,591	742,669	43.2	4,779,982
Colorado	3,860,230	2,132,493	89.3	1,587,936	893,102	44.6	12,516	7,054		3,649,027
Connecticut	871,263	431,585	10.0	1,448,790	722,376	15.0	1,124,229	547,740	9.4	1,475,189
Delaware	844,413	403,702	31.3	1,133,702	565,363	7.2				1,589,560
Florida	3,158,735	1,576,520	33.2	3,693,033	1,842,080	80.3	742,342	369,488	45.0	2,716,363
Georgia	4,295,781	2,032,397	236.0	4,647,443	2,323,722	238.8	3,356,271	1,678,136	112.1	7,178,535
Idaho	2,205,874	1,294,782	113.9	949,011	578,909	67.8	347,504	208,931	26.3	2,245,176
Illinois	8,732,235	4,344,678	195.7	6,103,922	3,049,972	97.2	4,089,850	2,023,425	116.9	4,950,678
Indiana	4,467,296	2,199,299	88.7	4,444,507	2,215,847	95.2	2,578,812	1,289,164	46.2	3,383,791
Iowa	3,897,721	1,823,441	190.7	3,775,417	1,682,388	111.9	1,463,686	691,100	67.2	2,881,769
Kansas	3,514,814	1,709,364	176.4	2,602,274	1,297,426	150.4	4,165,147	2,081,997	244.5	5,784,340
Kentucky	3,352,373	1,664,158	112.3	1,755,563	876,225	31.0	2,357,758	1,178,879	56.0	4,085,122
Louisiana	716,918	355,250	23.9	12,129,245	3,113,488	46.7	1,722,267	843,988	50.7	3,715,371
Maine	2,183,320	1,063,204	52.4	747,557	373,778	16.7	21,640	10,820	.5	1,310,311
Maryland	2,824,344	1,367,116	39.0	1,536,275	759,325	32.4	535,200	257,500	3.4	2,474,054
Massachusetts	3,134,614	1,564,618	25.1	726,621	362,466	5.7	1,734,634	864,024	15.0	3,816,039
Michigan	4,878,603	2,334,336	115.4	3,422,470	1,623,234	113.6	4,186,703	2,093,351	128.3	3,780,133
Minnesota	5,754,882	2,814,684	383.9	3,421,466	1,688,707	161.5	1,675,296	836,990	133.1	5,861,830
Mississippi	5,496,684	1,871,827	256.5	5,469,008	2,350,465	175.6	2,635,700	1,133,500	172.0	2,804,229
Missouri	3,262,387	1,625,018	140.6	4,823,420	2,307,167	122.3	3,931,348	1,520,192	165.8	6,323,727
Montana	3,228,277	1,823,217	189.8	2,327,788	1,319,699	115.9	1,553,848	879,846	156.5	4,976,352
Nebraska	4,717,201	2,314,660	409.0	3,728,606	1,722,813	360.3	3,177,075	1,585,646	430.2	3,756,372
Nevada	1,113,758	943,594	53.6	1,291,654	1,108,687	61.3	163,482	142,523	12.6	1,800,919
New Hampshire	823,550	405,045	27.3	711,819	348,783	15.5	456,360	225,447	8.8	1,309,698
New Jersey	858,420	419,454	7.1	5,562,408	2,780,254	42.2	708,800	354,400	8.0	2,296,092
New Mexico	2,215,677	1,342,931	167.1	1,560,214	972,735	60.6	205,572	125,158	59.0	2,246,136
New York	8,639,476	4,192,678	167.9	12,022,032	5,774,651	168.1	1,314,940	654,970	21.4	5,704,109
North Carolina	5,747,704	2,864,759	353.3	4,420,575	2,210,382	204.5	1,807,150	903,160	93.8	2,800,776
North Dakota	266,609	141,147	41.7	1,307,780	711,478	78.0	3,291,437	1,833,788	309.6	4,414,414
Ohio	5,943,593	2,912,343	84.3	8,710,932	4,332,092	64.9	7,489,851	3,744,740	86.5	6,120,580
Oklahoma	2,524,355	1,335,459	129.4	3,513,155	1,864,539	80.5	2,323,625	1,196,531	105.2	4,755,342
Oregon	2,666,441	1,595,619	108.2	3,449,146	1,931,162	112.8	536,894	313,580	41.9	2,188,239
Pennsylvania	9,656,244	4,762,068	113.7	5,493,574	2,601,802	47.7	5,670,139	2,787,502	68.8	5,884,663
Rhode Island	647,270	318,040	7.8	983,842	490,965	10.2	263,748	131,690	2.5	1,264,218
South Carolina	2,715,688	1,217,808	85.1	1,298,317	606,694	86.5	1,372,409	621,248	89.5	2,855,059
South Dakota	3,427,251	1,891,415	329.1	2,778,040	1,586,760	307.5	1,801,020	994,190	274.7	4,149,123
Tennessee	3,783,232	1,807,048	91.0	3,532,532	1,766,266	62.4	1,584,918	792,459	55.7	4,773,047
Texas	13,156,593	6,466,713	682.3	10,651,731	5,265,137	524.7	2,303,249	1,103,612	132.1	8,597,780
Utah	2,301,982	1,611,553	98.1	792,200	565,065	51.1	158,655	108,560	1.5	1,787,162
Vermont	737,383	348,555	18.4	711,393	355,507	22.7	740,844	366,394	18.1	559,599
Virginia	2,363,545	1,175,262	76.9	2,651,279	1,270,585	60.5	658,946	316,873	17.9	2,680,345
Washington	2,267,884	1,151,202	38.5	3,278,473	1,717,986	27.0	990,224	521,200	30.5	1,698,946
West Virginia	1,956,374	1,033,591	50.1	1,652,026	822,656	48.1	1,225,500	608,655	25.0	2,751,526
Wisconsin	5,171,809	2,539,964	187.9	5,021,815	2,464,880	153.2	55,948	25,805	2.4	4,450,008
Wyoming	1,501,001	917,048	141.3	1,618,252	1,020,311	162.7	391,470	236,637	44.1	1,457,899
District of Columbia	373,200	186,600	2.5	233,724	116,862	2.4	425,236	211,600	3.2	533,063
Hawaii	499,394	232,151	5.1	817,679	408,816	14.5	433,940	215,300	7.7	1,597,995
Puerto Rico	706,072	351,850	14.1	1,306,457	646,785	25.3	56,925	28,180	.2	908,605
TOTALS	176,673,052	90,520,351	6,672.2	171,152,050	83,634,069	5,015.1	81,644,869	40,561,107	3,652.8	170,529,382

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MARCH 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 615,939	$ 301,951	41.9	$ 960,807	$ 379,023	25.1	$ 450,750	$ 224,200	27.7	$ 632,777
Arizona	282,201	201,024	33.4	222,706	123,926	23.1				429,402
Arkansas	949,532	807,994	93.0	130,650	94,073	11.7	44,102	22,051	8.7	348,013
California	821,618	467,084	36.1	422,864	229,524	13.2	445,403	238,232	19.7	997,091
Colorado	900,353	475,952	31.3	247,932	104,109	7.2				278,316
Connecticut	172,310	72,417	2.9	100,760	50,197	.2	210,228	99,380	3.1	234,501
Delaware	80,623	39,067	17.5	69,537	34,768	7.8				309,475
Florida	885,834	438,113	31.3	29,916	14,958	.6	101,240	50,620	1.3	563,391
Georgia	341,690	163,919	44.2	184,992	92,496	16.9	471,231	235,616	44.0	1,278,473
Idaho	458,334	248,575	45.1	128,663	78,481	12.8				293,770
Illinois	1,282,835	625,708	78.8	1,105,035	496,705	62.6	864,950	417,475	33.0	733,174
Indiana	896,572	423,398	77.7	352,270	174,931	23.4	169,070	84,500	13.1	970,530
Iowa	348,453	167,023	128.4	1,141,640	543,885	125.5	1,139,084	536,655	265.9	812,477
Kansas	75,281	37,624	44.4	631,904	315,952	10.9	124,046	62,023	15.0	1,554,612
Kentucky	1,096,826	325,230	71.4	552,881	182,184	37.1	610,401	197,925	64.1	416,283
Louisiana	808,888	390,729	67.2	333,316	166,603	30.3	79,120	39,560	7.8	451,252
Maine	435,011	210,553	25.4	72,824	35,206	3.5	8,300	4,150	.5	142,001
Maryland	281,641	137,157	19.2	117,996	46,998	4.4				474,694
Massachusetts	373,212	185,203	9.2	284,559	140,915	5.9	257,813	127,759	7.1	541,986
Michigan	1,306,129	638,782	113.7	914,039	460,409	45.8	712,925	350,218	57.0	741,020
Minnesota	950,336	467,529	122.3	355,950	146,104	25.8	62,853	31,427	16.5	1,417,984
Mississippi	363,933	181,967	40.7	956,962	466,696	52.1	98,000	48,900	.5	716,477
Missouri	930,960	457,875	139.5	475,232	237,616	36.7	428,992	179,408	67.6	840,278
Montana	817,261	461,241	71.6	293,494	164,033	31.2	430,261	243,181	49.5	767,607
Nebraska	1,024,527	489,997	209.5	416,205	207,944	56.5	337,389	168,695	46.7	453,912
Nevada	164,206	134,299	25.0	197,591	169,920	25.5				228,342
New Hampshire	58,755	29,144	2.3	118,342	55,625	3.2	21,867	10,676	1.0	197,644
New Jersey	332,799	163,171	12.1	306,190	153,095	12.3	310,690	155,345	6.0	522,939
New Mexico	470,132	287,240	42.1	266,287	166,062	13.8	101,564	63,386	13.1	262,435
New York	1,820,821	879,099	91.3	1,613,175	775,643	44.8	537,841	203,721	12.7	797,640
North Carolina	1,069,398	531,728	106.2	620,243	312,733	54.6	338,390	159,915	28.2	394,504
North Dakota	115,841	61,370	8.3	62,121	34,895		150,963	80,915	3.8	1,012,290
Ohio	697,494	347,437	41.8	907,770	460,660	38.6	2,062,400	980,057	64.0	1,044,378
Oklahoma	260,138	133,815	10.5	744,506	395,457	51.3	202,390	107,007	22.1	1,039,633
Oregon	614,073	347,752	67.6	304,094	164,557	35.5	167,895	101,020	23.9	371,626
Pennsylvania	2,044,841	991,595	117.1	1,253,808	620,799	43.1	950,637	474,556	25.2	429,527
Rhode Island	93,827	46,890	2.2	186,882	93,441	2.0	43,173	21,560	1.6	95,049
South Carolina	578,846	234,056	56.9	297,040	112,595	25.9	358,820	134,229	35.1	307,605
South Dakota	16,229	8,937	4.1	14,680	9,712					1,280,913
Tennessee	818,717	351,682	31.7	146,416	73,208	.7	175,069	87,534	10.9	986,082
Texas	2,442,661	1,203,995	269.0	1,187,653	583,568	140.2	307,076	147,210	55.3	1,322,941
Utah	229,586	126,765	38.5	121,120	76,528	8.3				297,598
Vermont	138,763	63,954	5.6	164,292	50,336	6.3	167,400	52,029	5.8	83,270
Virginia	649,933	315,482	65.5	465,030	206,433	18.4	99,000	49,500	13.5	383,337
Washington	585,503	303,938	43.1	281,527	147,657	16.9	138,883	73,500	15.7	378,872
West Virginia	145,150	72,575	8.3	331,265	165,632	18.0	139,458	69,204	8.0	447,839
Wisconsin	883,504	439,012	34.5	321,053	160,190	4.7	379,319	179,040	4.4	833,468
Wyoming	470,702	286,620	26.0	329,482	195,846	23.5	53,943	34,052	15.5	189,322
District of Columbia	98,700	49,350	1.0	41,692	20,346	.5				76,554
Hawaii	193,033	96,211	6.2	273,208	137,108	8.6				159,806
Puerto Rico	30,220	14,440	2.2	244,134	119,025	12.9	55,188	27,140	2.1	109,023
TOTALS	31,524,171	15,936,669	2,714.8	21,302,735	10,448,807	1,279.9	13,808,124	6,573,571	1,116.7	29,652,133

U. S. GOVERNMENT PRINTING OFFICE 1940

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.
Report of the Chief of the Bureau cf Public Roads, 1935. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . . Official Inspection of Vehicles. 10 cents.
Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.
No. 191MP . . Roadside Improvement. 10 cents.
No. 272MP. . Construction of Private Driveways. 10 cents.
No. 279MP. . Bibliography on Highway Lighting. 5 cents.
Highway Accidents. 10 cents.
The Taxation of Motor Vehicles in 1932. 35 cents.
Guides to Traffic Safety. 10 cents.
An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.
Highway Bond Calculations. 10 cents.
Transition Curves for Highways. 60 cents.
Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.
No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.
No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.
House Document No. 272 . . . Toll Roads and Free Roads.
Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).
Report of a Survey of Transportation on the State Highways of Vermont (1927).
Report of a Survey of Transportation on the State Highways of New Hampshire (1927).
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).
Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).
Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.
Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.
Act III.—Uniform Motor Vehicle Civil Liability Act.
Act IV.—Uniform Motor Vehicle Safety Responsibility Act.
Act V.—Uniform Act Regulating Traffic on Highways.
Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MARCH 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 950,297	$ 946,632	10	2		$ 349,007	$ 348,924	8			$ 513,500	$ 513,500	2		5	$ 884,449
Arizona						518,060	515,813	4								400,746
Arkansas	184,063	184,017	3			672,928	666,423	3		3	332,008	330,926	4	1	8	807,458
California	1,156,384	1,151,614	8		1	1,331,046	1,164,844	8			55,247	54,747	1		6	1,885,803
Colorado	650,310	638,128	5		21	287,948	287,948	1		4	2,250	2,250			1	901,446
Connecticut	26,225	12,665				175,125	169,677		1	7	499,382	494,023	5			583,749
Delaware	7,839	7,839	2								96,433	96,433			16	568,269
Florida	428,094	428,094	2			196,653	192,154	2			23,903	23,903			6	1,438,727
Georgia	205,441	202,084	5		2	534,133	534,133	9	3		468,037	468,037	4	4	20	2,394,875
Idaho	204,578	172,785	3			215,380	214,413	2			135,426	105,800	2		3	499,709
Illinois	2,606,975	2,544,487	17	4	49	1,876,839	1,743,987	7		28	576,809	462,420	4	1	85	2,496,711
Indiana	770,618	770,618	3	1	61	706,108	706,108	3	2	51	193,324	193,324	1		58	1,390,943
Iowa	1,029,515	943,100	13		156	257,418	243,500	2		37	412,827	333,200	2		79	1,403,745
Kansas	934,280	931,180	11		8	658,242	658,242	8		1	454,856	454,856	5	2	11	1,197,144
Kentucky	594,500	589,647	8	4	5	820,701	820,702	7		1	240,150	240,150	4		21	813,688
Louisiana	493,976	493,958	4			460,321	406,826	5			627,885	570,153	11			789,393
Maine	336,835	334,299	3	2		209,733	209,733	2	1		13,768	13,768			5	416,551
Maryland	94,896	85,789	1		3	454,009	454,009	3		2	218,600	145,807		1	10	841,925
Massachusetts	400,519	399,288	2	2		122,047	122,014	2								2,335,881
Michigan	854,794	844,327	6	2	36	1,166,142	1,166,142	5	3	4	491,859	491,859	4		12	1,493,601
Minnesota	503,520	477,825	4	4	14	1,433,340	1,432,246	10	2	3	230,130	230,130	1	1	2	1,572,650
Mississippi	246,300	246,300	6			590,973	590,973	5	1		45,200	45,200	1			1,121,139
Missouri	402,019	400,714	2	1		1,419,062	1,419,062	9	2		861,759	584,775	1	3	1	1,795,880
Montana	850,426	850,426	9			217,073	101,991	3			92,599	90,817	2		4	609,806
Nebraska	704,212	703,613	19		30	861,548	861,548	7	2	10	79,004	79,004	1		14	820,634
Nevada	209,333	207,536		4	11	10,353	10,353			2						251,043
New Hampshire	102,433	101,921	7		1	103,328	103,292	3			77,921	77,921	1			392,776
New Jersey	262,682	262,682	1	1	1	743,959	743,959	3	2							1,755,390
New Mexico	72,824	71,821	2		1	188,425	188,425	2			138,907	138,907	1		4	620,095
New York	1,906,664	1,872,132	5	8		2,362,773	2,324,513	8	9		938,418	893,894	4	5	8	3,914,235
North Carolina	1,214,339	1,177,921	6	5	38	828,323	828,323	11	1	19	154,030	154,030	1	1	10	1,142,692
North Dakota	529,910	481,298	7	1		391,290	391,290	5			151,060	151,060	2		19	781,057
Ohio	543,740	528,740	8	1	3	1,504,853	1,433,901	8	2	1	977,340	977,340	5	1	1	3,372,589
Oklahoma	371,620	370,352	5		46	256,085	255,185	3	1	2	633,091	633,091	9		31	2,102,263
Oregon	173,428	171,720	2			318,721	218,374	3								568,943
Pennsylvania	1,480,102	1,268,203	2	2	1	1,167,064	1,167,064	10	2	1	350,000	350,000	1			5,568,697
Rhode Island	446,089	445,450	1	3		7,406	7,406									284,644
South Carolina	482,827	449,677	7	4	7	454,983	454,983	4	5		140,775	140,775	1	1	26	1,075,529
South Dakota	329,428	329,428	3	2	17	79,127	79,127	1	1		106,640	106,640	2		3	1,338,497
Tennessee	283,398	283,398		2	5	590,551	590,551	3	2							1,898,536
Texas	2,049,324	2,028,265	19	3	12	2,169,663	2,097,840	18		8	19,050	19,050			7	2,883,692
Utah	315,211	315,050	3		110	74,108	74,108			24	32,650	32,650			11	368,855
Vermont	32,135	27,295			7	206,402	206,402		2		2,969	2,969			1	260,937
Virginia	690,071	597,171	7	3	18	215,447	214,577	4			142,427	141,828	1	2	5	1,276,463
Washington	338,538	337,062	3	1	12	223,296	223,296	3	2		280,077	280,077	3	1	3	526,259
West Virginia	291,281	275,521	5		1	99,270	99,270	4			112,100	112,100	1		3	1,258,032
Wisconsin	905,123	885,062	9		10	828,576	782,479	7	2	1	468,167	439,834	2	1	11	1,385,774
Wyoming	136,007	135,851	1	1	8						85,910	85,910	1			632,365
District of Columbia						359,912	326,368	1								200,203
Hawaii	162,720	160,090	2	1		30,682	30,682	2			236,535	236,525	2		1	261,573
Puerto Rico	49,040	48,840	1			365,872	363,870	8			252,884	248,576	2			381,747
TOTALS	28,014,883	27,191,915	252	64	695	29,114,305	28,247,050	226	48	209	11,965,907	11,248,259	94	25	511	63,967,808

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 3 May 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

LOCAL IMPOSTS ON MOTOR VEHICLES IN MISSOURI

By JOHN H. LONG, State Manager, and BAILEY H. MAYES, Financial Manager, Missouri State-wide Highway Planning Survey

DETAILED data about the extent and incidence of State highway-user imposts have been collected and are being analyzed in 46 States in connection with the State-wide highway planning surveys. Information about similar local taxation is also essential for highway finance planning. Progressive changes in State and local tax policies and methods can be made only if adequate information is available to indicate the probable effect of any proposed changes on local governmental operation.

Over $65,000,000 was collected in Missouri through State and local motor-vehicle imposts during the 3-year period 1934–36.

License fees and motor-fuel taxes imposed by Missouri municipalities represented $10,658,000, or 16.1 percent, of the total motor-vehicle imposts. Of this, $4,742,000 was collected in local license fees and $5,916,000 in municipal motor-fuel taxes.

State imposts of $55,402,000, or 83.9 percent of the total, were derived from $24,687,000 of motor-vehicle registration and allied fees and $30,715,000 of motor-fuel taxes.

Local license fees or motor-fuel taxes were imposed in 289, or 37.2 percent, of the 778 incorporated places in the State. All places with a population of 5,000 or more imposed one or both of these taxes, so that 91.7 percent of the people living in incorporated places in Missouri were subject to these imposts.

Local motor-fuel taxes were not as commonly used as local license fees. In 17 municipalities the motor-fuel tax was the only local motor-vehicle impost, while in 223 places the license fee was the sole local motor-vehicle impost, and in 49 places both types of impost were employed.

During the 3-year period studied the average Missouri motor-vehicle owner living in a municipality where local highway-user taxes were imposed paid $3.28 in local license fees and $5.33 in local motor-fuel imposts. Payments in Kansas City and St. Louis in the same period for city license fees were $4.33 and $4.30 per vehicle, respectively, while city motor-fuel taxes amounted to $6.86 and $5.57 per vehicle, respectively.

Receipts from local motor-vehicle imposts did not increase as rapidly during the 1934–36 period as did receipts from State motor-vehicle imposts. Relatively greater ease of evasion of local taxes as well as other difficulties encountered in their collection appear to limit their effectiveness as sources of local revenue.

Information concerning the extent and incidence of State highway-user taxes in Missouri has been made available by the State-wide highway planning survey initiated in Missouri in 1935. Similar information about local taxes imposed on motor-vehicle owners in Missouri was collected for the 3-year period, 1934–36.

Local taxes on motor-vehicle owners as reported in this study are of two types—local license fees and local motor-fuel taxes.

The local motor-fuel taxes are comparable with the State motor-fuel taxes utilized by all States to raise revenue for highway purposes. Missouri is one of only seven States in which local governmental units levy taxes on motor fuels. Some counties in Alabama and Mississippi collect such taxes while cities and towns in Alabama, Florida, Louisiana, Missouri, New Mexico, and Wyoming collect motor-fuel taxes at varying rates. The maximum rates of local motor-fuel tax charged in the several States are shown in table 1.

Local motor-vehicle license fees are used much more extensively than are local motor-fuel taxes. Missouri is one of several States in which these taxes are levied. They are called variously license fees, wheel taxes, city tag fees, etc., and are levied against the motor-vehicle owners residing within the limits of the municipality levying the particular tax or fee. Complete data on this phase of local taxation in all States are not yet available.

Initial inquiry was made into the legal status of local motor-vehicle taxation in Missouri. Limits on the rates of municipal license fees were first provided in 1919. Since that time several changes in the law have been made and the State license fees, forming the basis upon which the local fees were charged, have been revised. Two conflicting provisions for limiting the rates charged by cities and towns were contained in the Missouri statutes from 1929 until 1935. Partly because of the changes in the laws and the confusion resulting from the conflicting provisions, a number of different schedules of maximum local license fees have been in effect.

TABLE 1.—*Maximum gasoline tax rates in States having local motor-fuel taxation in 1939*

State	Gasoline tax per gallon		
	State	County [1]	City or town [1]
Alabama	$0.06	$0.03	$0.02
Florida	.07		.01
Louisiana	.07		[2].02
Mississippi	.06	.03	
Missouri	.02		.01
New Mexico	.05		.01
Wyoming	.04		.01

[1] Highest rate reported in any county, city, or town.
[2] New Orleans repealed its 2-cent tax Oct. 1, 1939.

The various steps by which these rates have developed and the evolution of the present schedule are discussed in detail in the Appendix to this report, page 61. At the present time any city or town wishing to initiate such local taxation may establish license rates at not to exceed one-third of the present State rates. Cities that collected local license fees prior to 1933 are permitted to continue charging rates in excess of the later limits. The maximum legal rates are shown in table 2.

TABLE 2. *Comparative schedules of Missouri motor-vehicle license fees*

PASSENGER CARS

Basis of fee	State rate established 1933–34	Maximum legal rate for cities that adopted local license fees—Prior to 1933[1]	Maximum legal rate for cities that adopted local license fees—After 1933[2]
Less than 12 horsepower	$5.00	$2.50	$1.66
12 and less than 24 horsepower	8.50	4.25	2.83
24 and less than 36 horsepower	11.00	5.50	3.66
36 and less than 48 horsepower	20.00	7.50	6.66
48 and less than 60 horsepower	25.00	8.50	8.33
60 and less than 72 horsepower	31.50	10.50	10.50
72 or more horsepower	37.50	12.50	12.50

COMMERCIAL VEHICLES

Basis of fee	State rate established 1933–34	Prior to 1933[1]	After 1933[2]
Less than 2 tons	$10.50	$3.50	$3.50
2 and less than 5 tons	18.00	6.00	6.00
5 and less than 6 tons	27.00	9.00	9.00
6 and less than 7 tons	30.00	10.00	10.00
7 and less than 8 tons	36.00	12.00	12.00
For every ton or major fraction thereof in excess of 8 tons	15.00	5.00	5.00

[1] Equal to one-third of 1925 State license fee rates except for passenger cars having 12 horsepower to less than 24 horsepower per vehicle. Because of confusion in the law some cities charged rates higher than these legal rates. See Appendix for discussion of statutory development.

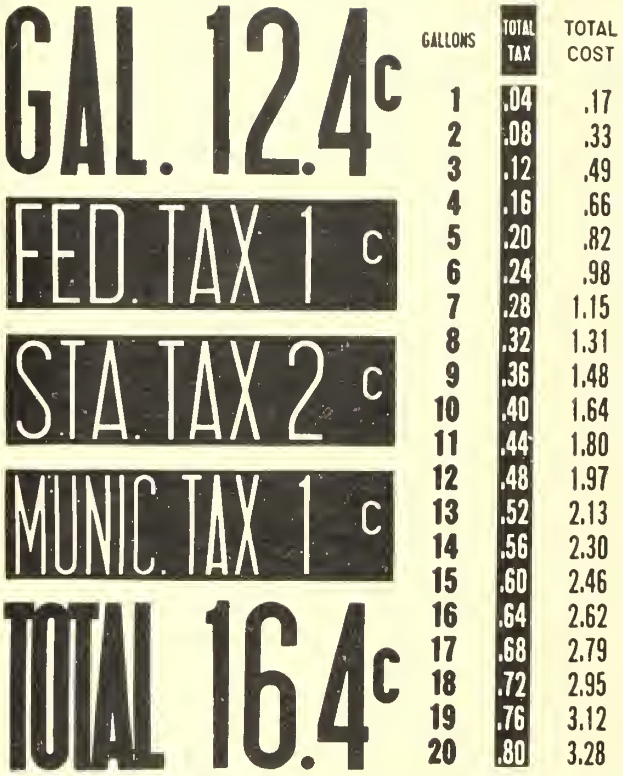
[2] Based on one-third of the State rates established in 1933–34.

No provision has been found in the Missouri statutes either specifically permitting or prohibiting cities and towns from levying taxes on gasoline. The 1929 Revised Statutes permit the larger cities to license and regulate manufacturers, merchants, and dealers of all kinds, but this provision has not been construed as specifically applicable to gasoline taxes levied against the consumer. Although it has been applied to gasoline dealers, it does not appear to have been actually intended as a gasoline or sales tax but rather as a license for the privilege of doing business. However, the amount of tax to be paid in the several cities that have used it has been determined by levying a certain amount of tax per gallon of gasoline sold. The tax thus works out practically to be a gasoline sales tax, even though it apparently should be considered strictly an occupation tax, the 1-mill to 1-cent-per-gallon rate being merely the method of determining the amount of the occupation tax to be paid.

Similar provisions in the statutes apply to the intermediate and smaller cities and towns as well as the larger places. Under these provisions, various municipalities have passed ordinances placing a gallonage tax on gasoline which, instead of being paid by the dealer out of his gross income, has been treated as a direct tax on the purchaser.

The general concept, that the tax was basically an occupation tax, was supported by the State Supreme Court in 1924. Nevertheless, the fact that the tax is generally regarded as a direct tax against the motorist is evidenced by the sales charts posted on gasoline pumps in cities where such a tax is collected. A typical chart is shown in figure 1.

It is apparent, then, that though the local licensing of motor vehicles in Missouri is provided for by State statutes, the status of the local motor-fuel taxes is somewhat poorly defined.



GAL. 12.4c

FED. TAX 1 c

STA. TAX 2 c

MUNIC. TAX 1 c

TOTAL 16.4c

GALLONS	TOTAL TAX	TOTAL COST
1	.04	.17
2	.08	.33
3	.12	.49
4	.16	.66
5	.20	.82
6	.24	.98
7	.28	1.15
8	.32	1.31
9	.36	1.48
10	.40	1.64
11	.44	1.80
12	.48	1.97
13	.52	2.13
14	.56	2.30
15	.60	2.46
16	.64	2.62
17	.68	2.79
18	.72	2.95
19	.76	3.12
20	.80	3.28

FIGURE 1.—TYPICAL SALES CHART POSTED ON GASOLINE PUMPS IN CITIES WHERE LOCAL GASOLINE TAXES ARE COLLECTED.

DATA OBTAINED FROM QUESTIONNAIRES AND HIGHWAY PLANNING SURVEY

Data for this study of local motor-vehicle imposts in Missouri were obtained from questionnaires returned by town and city officials throughout the State. Replies were received from practically all towns or cities that collected local highway-user taxes. Additional and supporting information was also obtained from the financial studies of the highway planning survey. The questions included on the forms sent out by the State Highway Department were:

1. Does your city collect a city gasoline tax?
2. At what rate per gallon?
3. What was the total amount collected for the fiscal years: 1934? __________; 1935? __________; 1936? __________.
4. Is the revenue credited to general revenue account?
5. If not, to what special use is the revenue put?
6. Does your city collect a motor-vehicle registration fee?
7. At what rates for cars? __________; for trucks? __________.
8. What was the total amount collected for the fiscal years: 1934? __________; 1935? __________; 1936? __________.
9. Is the revenue credited to general revenue account?
10. If not, to what special use is the revenue put?

Analysis of the replies by city and town officials to the questionnaires indicated that there was a wide range in the motor-vehicle license fees charged in the various places. For passenger cars the fees ranged from 25 cents to $12.50 per year, while for trucks the range was from $1.50 to $15. In some places, flat rates were charged; in others, the rates were based on the horsepower of the vehicles. In some places the local fees were assessed on the same basis as the State fees. One city reported a fee for trucks only. Some cities

TABLE 3.—*Incorporated places in Missouri which assessed local motor-vehicle license and motor-fuel taxes, 1934–36*

Population group	All incorporated places [1]		Places that reported assessment of local—							
			Motor-vehicle license fees				Motor-fuel taxes			
	Number	Population	Number	Percent	Population	Percent	Number	Percent	Population	Percent
Incorporated places having a population of—										
0–1,000	[2] 587	204, 687	[3] 122	20. 8	70, 444	34. 4	18	3. 1	9, 662	4. 7
1,001–2,500	[4] 118	181, 065	82	69. 5	129, 273	71. 4	21	17. 8	32, 198	17. 8
2,501–5,000	[5] 36	126, 123	31	86. 1	108, 723	86. 2	8	22. 2	29, 628	23. 5
5,001–10,000	21	151, 136	21	100. 0	151, 136	100. 0	8	38. 1	63, 496	42. 0
10,001–25,000	10	165, 060	10	100. 0	165, 060	100. 0	6	60. 0	100, 285	60. 8
25,001–100,000	4	197, 725	4	100. 0	197, 725	100. 0	3	75. 0	164, 271	83. 1
Kansas City	1	399, 746	1	100. 0	399, 746	100. 0	1	100. 0	399, 746	100. 0
St. Louis	1	821, 960	1	100. 0	821, 960	100. 0	1	100. 0	821, 960	100. 0
Total	778	2, 247, 502	272	35. 0	2, 044, 067	90. 9	66	8. 5	1, 621, 246	72. 1

[1] U. S. Bureau of the Census, 1930.
[2] Includes Des Peres (536) and Camdenton (850) incorporated since 1930.
[3] One place in this group assessed license fees on trucks only.
[4] Includes Olivette (1,180) and Afton (1,200) incorporated since 1930.
[5] Includes Flat River (2,631) incorporated since 1930.

reported that their ordinances provided for a license fee but that they had no collections because "the car owners did not want to pay it." In several places the wheel tax, or license fee, also included a 25-cent fee for collection. Table 3 shows the extent of use of local license fees in Missouri cities and towns.

Most towns and cities reported the same basic rates for both passenger cars and trucks. In 164 places the same rates were charged for all vehicles, while in 28 places the fees were graduated on the same basis for passenger cars and trucks. In some places these graduated fees were based on one-third, one-fifth, or 15 percent of the State rate; in others the rates charged were stated as varying from $1.50 to $2.50, or $2 to $5, or $3 to $6.50. One place reported that only trucks were charged a license fee and the rate was $2 for all capacities of trucks.

The rates of motor-fuel taxes charged in Missouri localities did not vary as greatly as did the motor-vehicle license fees, nor did as many places levy this form of tax. Table 3 also shows the number and percentage of incorporated places that levied local motor-fuel taxes in 1934, 1935, and 1936.

Almost all of the 66 communities that levied local motor-fuel taxes used either the ½- or 1-cent tax. One town, a place having less than 1,000 population, had a 1-mill rate and reported collections for the 3-year period of $453. Two towns reported a ¼-cent rate; while of the remaining 63 places, 32 used the ½-cent rate and 31 used the 1-cent rate. Both St. Louis and Kansas City charged the 1-cent rate. The weighted average motor-fuel tax rates, by population groups, are given in table 4.

TABLE 4.—*Average rates of motor-fuel tax [1] levied by Missouri municipalities, 1934–36*

Population group	Average motor-fuel tax per gallon			
	1934	1935	1936	1934–36
Incorporated places having a population of—				
0–1,000	$0. 0058	$0. 0057	$0. 0053	$0. 0056
1,001–2,500	. 0068	. 0070	. 0071	. 0070
2,501–5,000	. 0064	. 0064	. 0061	. 0063
5,001–10,000	. 0068	. 0066	. 0065	. 0066
10,001–25,000	. 0082	. 0083	. 0082	. 0082
25,001–100,000	. 0090	. 0092	. 0091	. 0091
Kansas City	. 0100	. 0100	. 0100	. 0100
St. Louis	. 0100	. 0100	. 0100	. 0100
Total	. 0093	. 0095	. 0095	. 0094

[1] Determined by weighting collections in each municipality by rate of tax per gallon.

Thus, of the 778 incorporated places in Missouri, it was found that 223 cities and towns assessed only a license tax, 17 assessed only a motor-fuel tax and 49 places assessed both types of tax. In all, 289 municipalities in Missouri derived some revenue from local motor-vehicle imposts during the period 1934–36. Further detail of this information, by population groups, is shown in table 5.

COLLECTIONS INCREASED EACH YEAR DURING 1934–36 PERIOD

Collections from these two taxes levied by local units averaged approximately 3½ million dollars per year for the period 1934–36 (table 6). Receipts from local motor-fuel taxes were higher than those from license fees in each of the three years. Receipts from both taxes increased each year. The individual reports showed only 3 places, with populations of 854, 3,507, and 4,485, that reported no license fee collections in 1936 but had reported collections for 1934 or 1935 or for both years.

Collections from local license fees in 1936 were less than in 1934 in 80 Missouri cities and towns. Of these 80 towns and cities, 28 were places having less than 1,000 population, 32 were places having 1,001 to 2,500 population, 11 had populations of 2,501 to 5,000, 6 had populations of 5,001 to 10,000 and 3 were in the 10,001 to 25,000 population group.

Two possible reasons for the decreased license fee collections in these 80 towns and cities as contrasted with increased registrations for the entire State may be suggested. A shift in motor-vehicle ownership to those areas and places where no fees were charged might have occurred, but complete data on actual motor-vehicle ownership by places during this period are not available to prove or disprove this. The decrease may have been due also to less efficient collection of the fees in some towns, because of increased opposition to and evasion of such charges by motor-vehicle owners.

Total collections from local motor-fuel taxes increased in each of the 3 years (except for the 1,001–2,500 population group). Only 18 places reported collections in 1936 that were lower than those in 1934, and 2 of these places had discontinued the tax since 1934. Four of the places that reported decreases had populations of 1,000 or less, 8 had populations between 1,001 and 2,500, and the other 4 had populations greater than 2,500. Three cities that reported no local motor-fuel taxes in 1934 had inaugurated them by 1936.

TABLE 5.—*Incorporated places in Missouri that assessed local motor-vehicle imposts, 1934–36*

Population group	All incorporated places [1]		Places that reported assessment of—															
			License fee only				Motor-fuel tax only				License fee and motor-fuel tax				Total			
	Number	Population	Number	Percent	Population	Percent	Number	Percent	Population	Percent	Number	Percent	Population	Percent	Number	Percent	Population	Percent
Incorporated places having a population of—																		
0-1,000	587	204, 687	111	18. 9	63, 494	31. 0	[2] 7	1. 2	2, 712	1. 3	[3] 11	1. 9	6, 950	3. 4	129	22. 0	73, 156	35. 7
1,001-2,500	118	181, 065	71	60. 2	112, 483	62. 1	10	8. 5	15, 408	8. 5	11	9. 3	16, 790	9. 3	92	78. 0	144, 681	79. 9
2,501-5,000	36	126, 123	23	63. 9	79, 095	62. 7					8	22. 2	29, 628	23. 5	31	86. 1	108, 723	86. 2
5,001-10,000	21	151, 136	13	61. 9	87, 640	58. 0					[4] 8	38. 1	63, 496	42. 0	21	100. 0	151, 136	100. 0
10,001-25,000	10	165, 060	4	40. 0	64. 775	39. 2					6	60. 0	100, 285	60. 8	10	100. 0	165, 060	100. 0
25,001-100,000	4	197, 725	1	25. 0	33, 454	16. 9					3	75. 0	164, 271	83. 1	4	100. 0	197, 725	100. 0
Kansas City	1	399, 746									1	100. 0	399, 746	100. 0	1	100. 0	399, 746	100. 0
St. Louis	1	821, 960									1	100. 0	821, 960	100. 0	1	100. 0	821, 960	100. 0
Total	778	2, 247, 502	223	28. 7	440, 941	19. 6	17	2. 2	18, 120	0. 8	49	6. 3	1, 603, 126	71. 3	289	37. 2	2, 062, 187	91. 7

[1] U. S. Bureau of the Census, 1930; see table 3 for revisions.
[2] One place reported tax rate but no collections.
[3] In one place no license fee collections were reported, in another no gasoline tax collections, and in a third no collections from either tax were reported.
[4] Gasoline tax discontinued in one place after 1934.

TABLE 6.—*Annual collections of local motor-vehicle imposts in Missouri by population groups, 1934–36*

Population group	Annual collections in Missouri from—											
	Local license fees in—				Local motor-fuel taxes in—				Total local motor-vehicle imposts in—			
	1934	1935	1936	Total 1934–36	1934	1935	1936	Total 1934–36	1934	1935	1936	Total 1934–36
Incorporated places having a population of—												
0-1,000	$14, 863	$15, 922	$18, 167	$48, 952	$5, 979	$6, 063	$11, 145	$23, 187	$20, 842	$21, 985	$29, 312	$72, 139
1,001-2,500	46, 867	48, 375	49, 318	144, 560	33, 910	30, 229	32, 235	96, 374	80, 777	78, 604	81, 553	240, 934
2,501-5,000	50, 283	52, 677	51, 757	154, 717	36, 915	36, 969	39, 670	113, 554	87, 198	89, 646	91, 427	268, 271
5,001-10,000	95, 762	92, 773	101, 967	290, 502	63, 917	65, 030	68, 076	197, 023	159, 679	157, 803	170, 043	487, 525
10,001-25,000	111, 368	111, 215	117, 083	339, 666	107, 413	108, 883	115, 037	331, 333	218, 781	220, 098	232, 120	670, 999
25,001-100,000	135, 324	122, 611	149, 337	407, 272	144, 714	168, 295	172, 686	485, 695	280, 038	290, 906	322, 023	892, 967
Kansas City (399,746)	352, 271	365, 503	379, 825	1, 097, 599	552, 959	582, 401	605, 849	1, 741, 209	905, 230	947, 904	985, 674	2, 838, 808
St. Louis (821,960)	717, 117	738, 217	803, 981	2, 259, 315	626, 749	1, 123, 162	1, 177, 587	2, 927, 498	1, 343, 866	1, 861, 379	1, 981, 568	5, 186, 813
Total	1, 523, 855	1, 547, 293	1, 671, 435	4, 742, 583	1, 572, 556	2, 121, 032	2, 222, 285	5, 915, 873	3, 096, 411	3, 668, 325	3, 893, 720	10, 658, 456

TABLE 7.—*Comparison of population, registration, and State and local motor-vehicle imposts in Missouri, 1934–36*

Population group	Percentage of—							
	Total population in incorporated places [1]	Total motor-vehicle registration in incorporated places [2]	Local motor-vehicle imposts collected, 1934-36			State motor-vehicle imposts paid by vehicle owners of incorporated places, 1935 [3]		
			License fees	Motor-fuel taxes	Total local imposts	License fees	Motor-fuel taxes	Total State imposts [4]
Incorporated places having a population of—								
0-1,000	9. 1	11. 1	1. 0	0. 4	0. 7	9. 7	8. 8	9. 6
1,001-2,500	8. 0	9. 8	3. 1	1. 6	2. 2	8. 6	8. 0	8. 4
2,501-5,000	5. 6	7. 0	3. 3	1. 9	2. 5	6. 5	6. 0	6. 5
5,001-10,000	6. 7	8. 0	6. 1	3. 3	4. 6	7. 7	7. 6	7. 7
10,001-25,000	7. 4	8. 2	7. 2	5. 6	6. 3	8. 1	7. 7	7. 8
25,001-100,000	8. 8	8. 3	8. 6	8. 2	8. 4	8. 2	9. 6	9. 1
Kansas City (399.746)	17. 8	15. 5	23. 1	29. 5	26. 6	16. 3	17. 4	16. 6
St. Louis (821,960)	36. 6	32. 1	47. 6	49. 5	48. 7	34. 9	34. 9	34. 3
Total	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0

[1] 1930 U. S. Census. Includes all incorporated places whether they collected local motor-vehicle taxes or not.
[2] Based on highway planning survey analysis for 1935: includes all incorporated places.
[3] Based on highway planning survey analysis.
[4] Includes miscellaneous fees in addition to license fees and motor-fuel taxes.

The percentage relations of the amounts shown in table 6 and comparison with the motor-vehicle registrations and the populations of the various groups in the State are shown in table 7. These comparisons emphasize the great influence of the population concentrations of Kansas City and St. Louis.

During the 3-year period, receipts from similar highway-user taxes levied and collected by the State amounted to $55,402,000, compared with the total local collections of $10,658,456. The annual receipts from State highway user-taxes are shown in table 8. The relative magnitude of local and State collections for each of the 3 years is also shown in figure 2.

LOCAL COLLECTIONS ABOUT TWO-FIFTHS OF STATE COLLECTIONS IN SAME LOCALITIES

Another desirable comparison is that between the amounts of State taxes and local taxes paid by motor-vehicle owners in only those places that assessed local highway-user taxes. In the highway planning survey a study of motor-vehicle allocation determined the approximate average amounts of State (highway-user)

Table 8.—*Collections from State motor-vehicle and motor-fuel taxes in Missouri, 1934–36* [1]

Year	Collections from State—		
	License fees [2]	Fuel taxes [3]	Total
1934	$7,346,000	$9,682,000	$17,028,000
1935	8,353,000	9,845,000	18,198,000
1936	8,988,000	11,188,000	20,176,000
Total	24,687,000	30,715,000	55,402,000

[1] State Motor-Vehicle Receipts, 1934, 1935, 1936; Disposition of State Motor-Fuel Tax Receipts, 1934; State Motor-Fuel Tax Receipts, 1935, 1936—U. S. Public Roads Administration.

[2] Includes other receipts in connection with the registration of vehicles, such as receipts for chauffeur's permits and certificates of title.

[3] Includes other receipts in connection with the administration of the motor-fuel tax.

taxes paid by motor-vehicle owners in various population groups. This study also provided data showing the approximate numbers of vehicles in the several population groups and consequently the average number of persons per vehicle. Using these basic determinations it is possible to compute the approximate amount of State highway-user taxes paid by motor-vehicle owners resident in the localities that also levied local motor-vehicle and motor-fuel taxes. A summary of these computations is given in table 9.

Table 9.—*Approximate collections of State motor-vehicle fees and motor-fuel taxes in Missouri municipalities that levied local highway-user taxes, 1934–36*

Year	Approximate collections from State—		
	License fees	Fuel taxes	Total
1934	$4,279,513	$4,336,920	$8,616,433
1935	4,756,408	4,448,914	9,205,322
1936	5,156,223	4,692,758	9,848,981
Total	14,192,144	13,478,592	27,670,736

Comparison of tables 6 and 9 indicates that during the years 1934–36, Missouri localities that levied motor-vehicle license fees and motor-fuel taxes collected approximately 33 percent as much in local license fees and approximately 44 percent as much in local fuel taxes as the State collected in similar taxes in the same localities. Comparison of tables 8 and 9 indicates that in places that levied local taxes there was collected 57 percent of the State motor-vehicle license fees and 44 percent of the State motor-fuel taxes collected from 1934–36.

Two further interesting comparisons can be made using 1930 census data for the populations of the various places that levied local motor-vehicle and motor-fuel taxes and the determinations of the highway planning survey for the approximate numbers of vehicles in the various population classifications. These two comparisons concern the collections per person and per vehicle in places of various sizes. By disregarding fluctuations over the 3-year period and by basing average per-capita and per-vehicle collection computations on the total collections in each of the population groups, on the total population involved and on the total number of vehicles involved, the comparisons shown in figures 3 to 6, inclusive, have been made.

It will be seen in figures 3 and 5 that average collections of local motor-vehicle license fees, on both a per-capita and a per-vehicle basis, vary consistently with the size of the population group involved. The regularity of this variation is most pronounced for the per-vehicle comparison. The averages for all places are larger in both cases than the averages for any group of places except Kansas City and St. Louis.

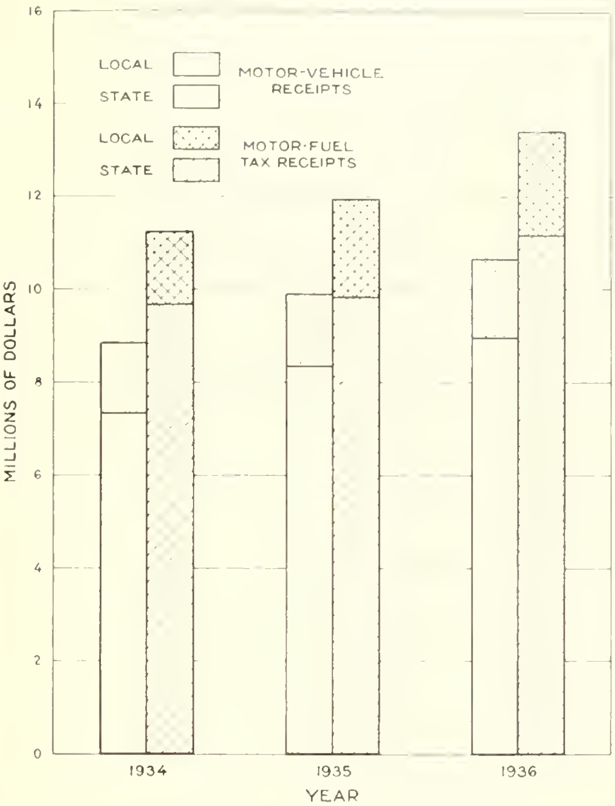


Figure 2.—Receipts From State and Local Highway-User Taxes in Missouri, 1934–36.

Figures 4 and 6 indicate no significant trends concerning the relation of local motor-fuel tax collections to the size of the places involved. Figure 6 is, however, significant in that it shows per-vehicle collections of motor-fuel taxes in Kansas City and St. Louis to have been higher than those for any other population group in the State. Two possible explanations for this condition, aside from the slight differences in tax rates given in table 4, are suggested. While motor-vehicle fees are strictly a measure of local contributions, the motor-fuel tax is not so accurate a measure. Motor-fuel taxes are collected from all persons purchasing motor fuel in the respective places regardless of whether or not the motor fuel is used in vehicles owned in those communities.

EVASION OF LOCAL GAS TAXES INDICATED

Kansas City and St. Louis, as urban centers of the State, attract large numbers of vehicle owners from elsewhere in the State and from other States and collect from them a certain amount of motor-fuel taxes in the normal course of such travel. Highway planning survey data indicate that the average urban-owned vehicle is driven a greater mileage each year than is the average car owned by residents of the smaller cities and unincorporated areas. Thus, using more motor fuel to accomplish such travel, the vehicle owners of those cities would contribute proportionately greater local fuel-tax revenues than would the vehicle owners of other cities, if gasoline were purchased within the city limits.

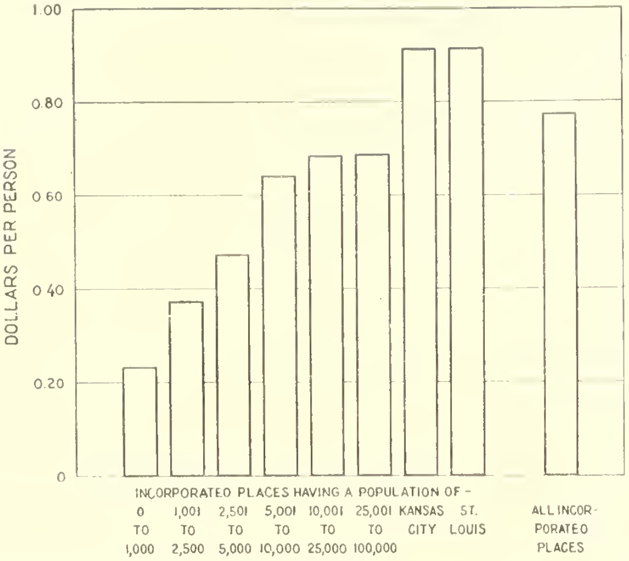


Figure 3.—Average Collections per Person of Local Motor-Vehicle License Fees in Missouri, 1934–36.

Comparisons between State and local motor-fuel tax collections, especially in Kansas City and St. Louis, are of interest. Highway planning survey analyses indicate that in 1935 Kansas City motor-vehicle owners contributed $1,247,232 and St. Louis motor-vehicle owners contributed $2,502,517 in State gasoline taxes, on a 2-cent tax rate. If the State rate had been only 1 cent per gallon and gasoline consumption had remained the same, contributions from motor-vehicle owners in these two cities to the State motor-fuel tax revenues would have been $623,616 and $1,251,258, respectively.

These amounts may be compared with the actual collections by these two cities from the local 1-cent-per-gallon taxes. In 1935 local gasoline tax collections were $582,401 and $1,123,162, respectively. These amounts were 6.6 and 10.2 percent less, respectively, than comparable contributions to State gasoline taxes.

These differences may be partly accounted for by the fact that Kansas City and St. Louis vehicle owners buy some of their gasoline on trips away from their respective cities and thus avoid paying local taxes. At the same time residents of other places traveling to these two cities very probably purchase some gasoline within the city limits and thus offset the losses occasioned by residents buying some of their gasoline elsewhere.

To these two considerations must be added a third—that some gasoline used in these two cities escapes local taxation because it is purchased at filling stations located outside the city limits. Advertisement is often given to the fact that a station is outside the city limits and that the purchaser of gasoline can avoid the city tax on his purchases. Figure 7 shows two views of a filling station located outside a Missouri city in which local motor-fuel taxes are collected. Numerous other stations near the city limits similarly advertise the fact that no city tax is collected. A considerable portion of the difference between city and State collections cited above can probably be credited to this factor.

Questions 4, 5, 9, and 10 of the questionnaire sent to the cities and towns in Missouri were designed to determine the uses made of the local highway-user revenues. Not all of the cities and towns replied adequately to these particular questions but the answers from 260 places were of such a nature that a partial analysis could be made. In addition, more detailed analyses of the Kansas City and St. Louis data for the 3 years were made. The percentage distributions of the funds for each of the 3 years studied in the 260 places reporting, exclusive of Kansas City and St. Louis, are shown in table 10.

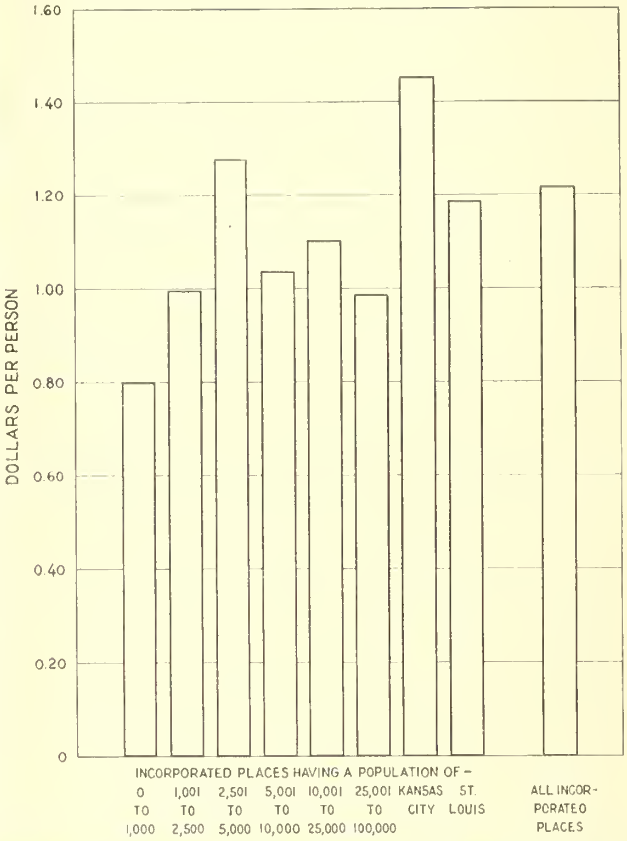


Figure 4.—Average Collections per Person of Local Motor-Fuel Taxes in Missouri, 1934–36.

Table 10.—*Distribution of local motor-vehicle and motor-fuel taxes in 260 Missouri cities and towns, 1934–36*

Year	Percentage of local highway-user taxes—		
	Paid to general revenue fund	Paid to street maintenance fund	Total
1934	52.3	47.7	100
1935	54.6	45.4	100
1936	53.9	46.1	100
Average	53.6	46.4	100

It appears reasonable to expect that in many places appropriations from the general revenue fund were used for street and alley improvements, so that the percentage of the total collections actually used for street purposes was undoubtedly larger than the 46.4 percent shown in table 10. Highway planning survey data indicate that in some places money collected from motor-vehicle owners in the form of local motor-vehicle and motor-fuel imposts was not spent entirely for street purposes, but in other places much more was used on the streets than was collected locally from motorists.

In one particular instance a city reported a collection from local highway-user taxes of $15,000 in 1934 for its motor-vehicle license fund. Disbursements included

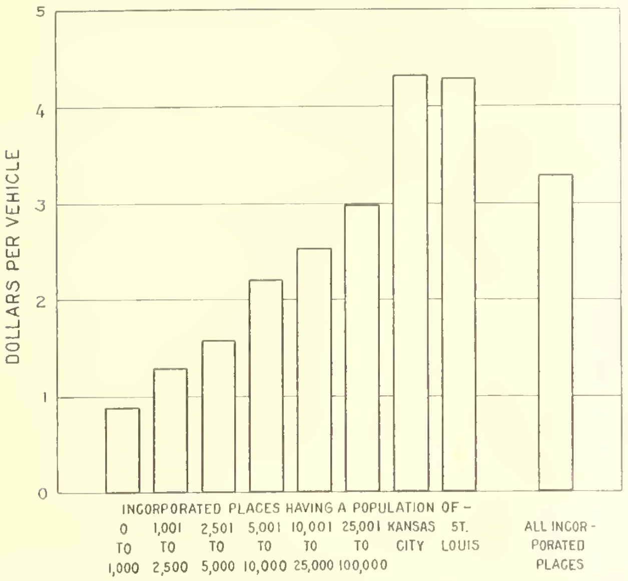


Figure 5.—Average Collections per Vehicle of Local Motor-Vehicle License Fees in Missouri, 1934–36.

a transfer to the general fund of $700 in addition to street expenditures of $6,900. But there were payments amounting to $27,000 for streets and bridges from the city's general fund. A similar condition existed in that city in 1935 when there were payments of $23,000 for streets from the general fund in addition to net expenditures from the motor-vehicle license fund of $12,300, as compared to net receipts from local highway-user taxes of $12,000.

Both Kansas City and St. Louis used all of their revenues from locally imposed motor-vehicle taxes for street purposes, though for each of the 3 years the distributions to funds were actually as shown in table 11. The Kansas City ordinances provide that all gasoline tax money collected locally shall be used and expended for the repair, upkeep, and maintenance of the public streets and highways of that city. It is also provided in the city charter that:

> Subject to the right of the city council to appropriate not to exceed 3 percent thereof to the fireman's pension fund, all sums derived from license taxes collected by the city * * * shall be appropriated and used exclusively for the maintaining, adorning, constructing, and repairing and otherwise improving the parks, parkways, boulevards or other highways.

Table 11.—*Distribution of receipts from Kansas City and St. Louis local highway-user imposts, 1934–36*

Year	Distribution of receipts in—			
	Kansas City to—		St. Louis to—	
	General revenue fund	Park districts [1]	Municipal fund	Interest and sinking fund
1934	$555, 981	$349, 249	$1, 299, 639	$44, 227
1935	585, 807	362, 097	1, 304, 654	556, 725
1936	610, 126	375, 548	1, 396, 182	585, 386
Total	1, 751, 914	1, 086, 894	4, 000, 475	1, 186, 338

[1] Street and boulevard improvements.

SMALL PLACES RECEIVE PROPORTIONALLY GREATER BENEFITS FROM STATE HIGHWAY EXPENDITURES

Two further analyses and comparisons were made on the basis of available data. The first concerned the amount that would have been raised in any year if all

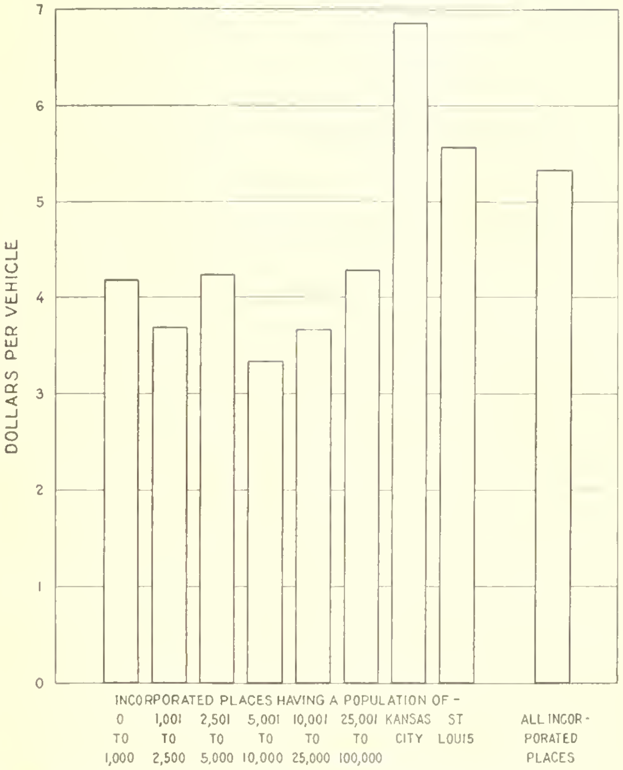


Figure 6.—Average Collections per Vehicle of Local Motor-Fuel Taxes in Missouri, 1934–36.

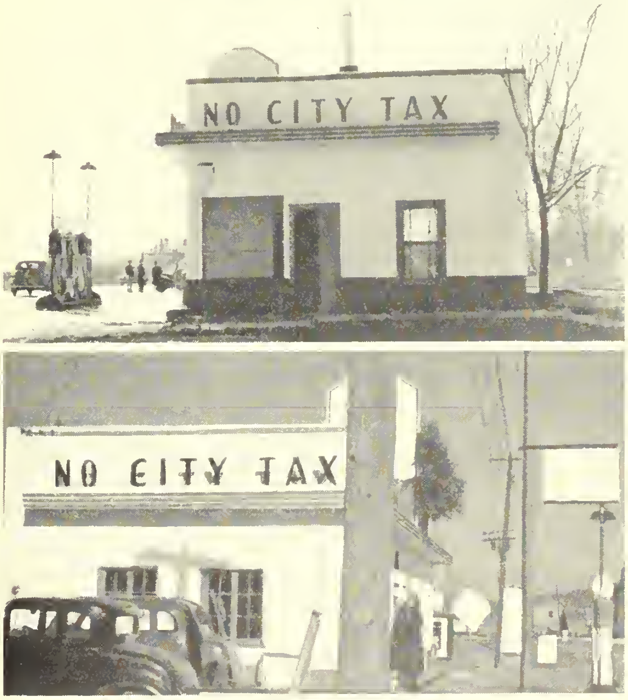


Figure 7.—Two Views of a Filling Station Located Outside a Missouri City. Top, View as One Approaches the City; Bottom, View as One Leaves the City.

incorporated places in Missouri had levied and collected local license and motor-fuel taxes equal to the average collections in those places of the same population classification that levied such taxes.

This comparison can be made on the basis of either the average receipts per person or the average receipts per vehicle in the towns and cities where the taxes were collected. The comparison was made for 1936 and is shown in table 12. The computations were based on the 1930 population figures of the places affected and results of the 1935 motor-vehicle allocation study of the planning survey which determined the approximate number of vehicles in the various population groups. The average license fee collected in 1936 by 122 places having populations of 1,000 or less was $0.258 per person, and the average motor-fuel tax collected by 18 places in the same population group was $1.153 per person. The total population of all the 587 places in the population group was 204,687 which, multiplied by each of these per-person figures, gives $52,809 and $236,004, respectively. These, added together, give the $288,813 shown in the third column of table 12.

This analysis indicates that the adoption of such a policy would have produced, when the population and registration basis results are averaged, an increased total annual collection of only $795,587. More than $600,000 of this increase would have been produced in the places having less than 5,000 population.

TABLE 12.—*Estimated amounts that all Missouri cities and towns would have collected in 1936 in local highway-user imposts if rates were based on average receipts of those places that levied such taxes*

Population group	Total local motor-vehicle imposts, 1936	Estimated collections for all places based on—		
		Population	Motor-vehicle registration	Average
Incorporated places having a population of—				
0-1,000	$29,312	$288,813	$335,769	$312,291
1,001-2,500	81,553	250,413	276,161	263,287
2,501-5,000	91,427	228,914	234,576	231,745
5,001-10,000	170,043	264,035	257,799	260,917
10,001-25,000	232,120	306,352	286,651	296,502
25,001-100,000	322,023	357,091	357,556	357,323
Kansas City (399,746)	985,674	985,674	985,674	985,674
St. Louis (821,960)	1,981,568	1,981,568	1,981,568	1,981,568
Total	3,893,720	4,662,860	4,715,754	4,689,307

It was previously pointed out that expenditures for city streets in many cities and especially in the larger cities exceed their receipts from local motor-vehicle imposts. In this same connection, it may be noted that statutory provision [1] for State highway construction in the smaller places has been made as follows:

> Any State highway which passes through a municipality having a population of less than 2,500 according to the last United States census preceding such construction, shall be constructed through such municipality, and any highway which passes through a municipality having more than 2,500 population, according to such census, shall be constructed through the portions of the municipality where the houses abutting such roadway are not less than 200 feet apart on the average; *Provided, however,* That in either case the State shall not pay for road surfacing wider or of a higher type of construction than the road as constructed up to the boundary of such municipality.

It is thus apparent that the smaller places can receive proportionally greater benefits from the continued expenditure of State highway funds than can the larger towns and cities. Table 13 shows the expenditures made in 1934 by Missouri towns and cities on their own streets. Additional data on the expenditure of other funds in the various cities, based on highway planning survey results, show clearly that in the larger cities the State expenditures are a much smaller percentage of the total street expenditures than they are in the smaller places.

Direct comparisons of the amount of local highway-user taxes collected to the local street expenditures cannot be made because so many of the smaller cities did not collect local motor-vehicle taxes, and expenditure data for those that did cannot be readily separated from the data for those places that did not collect these local taxes. Though 22 percent of the places having 1,000 population or less collected local highway-user taxes, the amount they raised in 1934 was only 10.6 percent of the street expenditures of all places in that group. In the 2,501 to 5,000 population group, 86.1 percent of the places levied local highway-user taxes and collected a total amount equal to only 37 percent of the street expenditures of the entire group. In Kansas City receipts from local highway-user taxes in 1934 were 68.7 percent of the city's expenditures for streets, while in St. Louis the collections equaled only 32.3 percent of the total street expenditures by the city.

TABLE 13.—*Expenditures for town and city streets in Missouri by population groups in 1934* [1]

Population group	Expenditures for local streets by—	
	Cities and towns	State highway department
Incorporated places having a population of—		
0-1,000	$197,300	$324,400
1,001-2,500	268,000	168,100
2,501-5,000	235,900	640,500
5,001-10,000	330,400	321,300
10,001-25,000	372,400	235,000
25,001-100,000	360,100	324,200
Kansas City (399,746)	1,316,900	576,600
St. Louis (821,960)	4,164,600	119,100
Total	7,245,600	2,709,200

[1] Does not include debt retirement.

In addition to the amounts shown in table 13, large amounts were spent in 1934 by Missouri towns and cities for the retirement of debt originally incurred for street purposes. Most of the $5,667,600 of debt retirement so effected was accounted for by St. Louis where the amount retired was $5,422,000. In all places having populations of 10,000 or less, the total street debt retired in 1934 amounted to only $61,100.

SUMMARY

The data collected and analyzed in this study have importance in the general problem of street and highway planning in Missouri insofar as they indicate the degree of reliance of municipalities on these revenues and the relation of those imposts to State fees and taxes imposed on similar bases and for similar purposes. Any proposed statutory revision that would affect these taxes must be carefully analyzed to determine what effects on street and highway funds and on street and highway work would result.

This study has indicated that receipts from local imposts are not, for population groups as a whole, sufficient to meet the expenditures by cities and towns on their own streets. If present average impost rates were extended to all cities and towns in the State, the amounts so raised would fail to equal the amounts now spent for streets by the towns and cities of the various population groups, with the exception of the places having populations of 1,000 or less.

(*Continued on page 61*)

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[1] Laws of Missouri relating to roads, highways, and bridges (Revised Statutes 1929); sec. 8133.

SOME EXPERIENCES WITH EXPANSION JOINTS IN CONCRETE PAVEMENTS

RUBBER AND RUBBER COMPOUNDS USED AS A FILLER OR SEAL

BY THE DIVISION OF CONSTRUCTION, PUBLIC ROADS ADMINISTRATION

Reported by **ANDREW P. ANDERSON**, Highway Engineer

CONSIDERABLE attention has been directed during recent years toward the development of more satisfactory materials for filling and sealing expansion joints in concrete pavements. One of these newer developments involves the use of rubber in various forms or in combination with various other materials. From time to time and at various places these new rubber compositions or combinations have been given more or less extensive trials on actual construction jobs. A good many of these materials have now been in service long enough to indicate something of their probable life and behavior under traffic. The results obtained from these tests, however, have apparently never been fully accumulated for comparative study. A fairly extensive inquiry has therefore been made as to the extent of use of these materials on Federal-aid work and the results indicated to date.

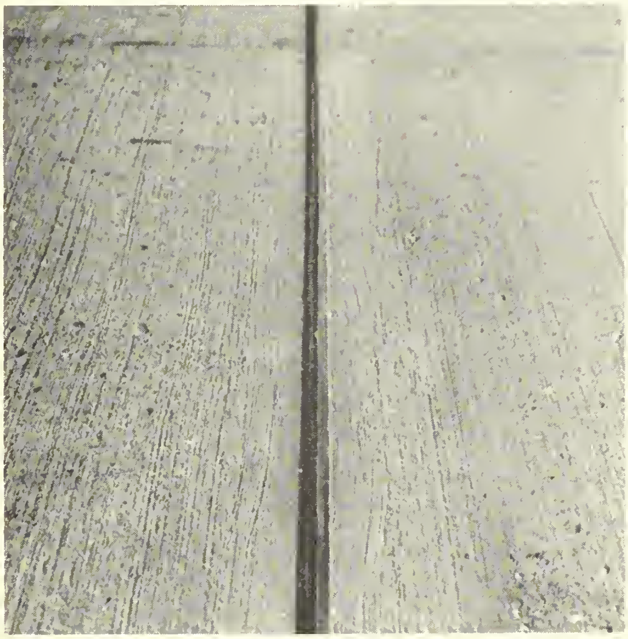
The returns include observations on more than a hundred projects distributed throughout 28 States where one or more of these joint fillers have been in service for a period of at least a year. The cases in which the results have been disappointing are far more numerous than those which have been satisfactory, as is frequently true during the period of development of new materials.

A fully satisfactory material for filling or sealing expansion joints in concrete pavements must be very elastic and fully able to accommodate itself to the movement of the slab without appreciable extrusion, and must always maintain such close contact with the ends of the concrete slabs as to prevent the entrance of either water or other foreign materials. It must be able to maintain these qualities for a considerable period of time without appreciable impairment, be easy to install, and be of relatively low cost.

No material has as yet been introduced which fully satisfies all these requirements. The standard bituminous filler is easy to pour or install, is of low first cost, but it is lacking in the required elasticity. As the slabs expand the filler extrudes, forming a ridge or bump for every passing vehicle. When the slabs contract the filler usually fails to take up the additional joint space, enabling water and foreign materials to gain entrance.

Rubber is a very elastic material but its cost is relatively high and it tends to deteriorate under the severe service conditions imposed by the combination of weather and traffic. Sponge rubber has been used for about 10 years as a joint filler. Recent inspections on 16 jobs indicate that where fully protected by a tight seal the behavior of this material has in general been satisfactory for a considerable period. But where no seal, or an ineffective seal, has been provided the sponge-rubber filler after about 3 years' service generally shows appreciable deterioration and loss of elasticity.

In Florida about 125 premolded, sponge-rubber joint fillers were installed on one job in the fall of 1936 and early part of 1937. When inspected on November 16, 1939 it was found that the poured bituminous seal was gone from most of the joints. In these the unprotected rubber material had oxidized, become brittle, and begun to split and pull away from the concrete. This weathering action was found to have reached an average depth of about 2 inches. On the other hand, where the rubber had been continuously protected by the bituminous seal it remained alive to the top and made proper contact with the concrete.

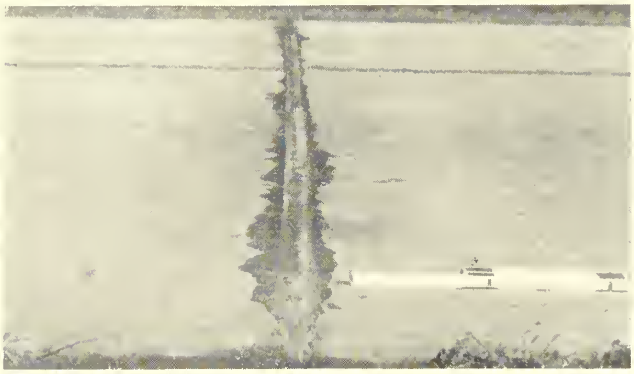


An Expansion Joint That Approximates Design Requirements.

SPONGE-RUBBER FILLERS OFTEN FOUND TO HARDEN AND SHRINK

In Indiana 1-inch premolded, sponge-rubber fillers were used in the expansion joints on a Federal-aid job in 1931. The joints were sealed but it was soon found that the cut-back asphalt then used as a seal and crack filler seemed to act on the sponge rubber in the joint and make it quite soft and more spongy. Therefore, on work in 1934, 1935, and 1936 the joints in which rubber was used as a filler were not sealed. A careful examination in November 1939 on seven of these jobs showed: First, that this type of premolded, sponge-rubber filler had taken on a permanent set in about 3 years of actual use; and second, that where the rubber was not protected by a good seal but was exposed to the weather, it soon became hard, lifeless, and shrank so that it no longer completely filled the joint space.

Careful measurements on the seven projects showed that the thickness of the existing rubber fillers at their narrowest places varied from one-half to seven-eighths inch as compared with the original 1-inch width. Furthermore, while some set was found in the rubber filler on the early construction in 1931 and years immediately following in which the expansion joints were sealed, the permanent set was much less where the seal had been maintained than for the later construction where no seal was used or on the old construction where the seal of the joints had been broken.



Appearance of Bituminous-Filled Joint After the Filler Has Extruded Due to Pavement Expansion.

During more than 10 years of experience with sponge rubber joints in California highways it has been found that this material soon loses its resiliency and is in time compressed to a fraction of its original thickness, leaving the joint open when the slab contracts. Quite often the compressed filler is shredded and drawn out of the joint under the action of traffic.

In Minnesota five premolded, sponge-rubber expansion joint fillers were used in a pavement built in 1930. All five joints were sealed with hot-poured bituminous material. On November 9, 1939, it was found that where the seal had been poor or ineffective, the upper part of the sponge rubber was badly deteriorated, cracked, and lifeless. The lower part, however, was in good physical condition generally, and showed no loss of resilience. Where the bituminous seal had given protection the oxidation, with resulting hardening, cracking and loss of material, was much reduced.

In Nevada sponge rubber was used as a filler for all the expansion joints on a 1.22-mile job in 1933. This material gave very good service in 1934, 1935, and 1936. During exceptionally hot weather in 1937, however, the filler extruded so badly that about half of the total amount in the joints was worn away by traffic and completely lost. When the joints were refilled and sealed that fall, it was noticed that the sponge rubber had largely lost its resiliency and had become somewhat brittle.

In order to overcome the objection of high cost of straight rubber products, various compounds have been developed in which cheaper materials are added to give bulk and so decrease the total cost. The more common of the ingredients added to the rubber latex are ground cork, ground or treated mica, mineral powders, tar, asphalt, etc. In these compounds rubber or rubber latex is the material relied on to supply the properties of elasticity and to give firm adhesion to the ends of the concrete slabs. Sometimes these compounds are used as a complete filler of either the poured or premoulded type but more frequently they are used as a seal to cheaper materials which occupy the greater part of the joint space, or to seal joints of the premolded or metal type.

Included in such compounds are a large number of proprietary products, the exact compositions of which are frequently not revealed. Some of these products are poured cold; others must be heated. Some are sold all prepared and ready for application, while others must be combined on the job before they are ready for use. For some the application is completed in one operation, while others require two or more operations, such as first priming the edges of the concrete before pouring the main seal. Because of the many variations definite comparisons of the products are difficult, frequently impossible. Some typical examples chosen from observations on 62 jobs will, however, be given of the successes and failures experienced with this type of joint filler.

Left, a 2-Inch Poured Rubber Composition Seal in Good Condition; Right, a Poured Rubber Composition Seal That Has Extruded.

BOND BETWEEN JOINT FILLER AND PAVEMENT OFTEN INADEQUATE

On one job in Connecticut completed in 1936, a rubber compound was used to seal both the expansion and dummy joints. In November 1939 the original seal was entirely gone from all the expansion joints. In the dummy joints the original seal was still in place but was badly cracked and disintegrated. On this job trouble with the joint seal began immediately after the joints were poured. The bond secured between the filler and the concrete was apparently insufficient to prevent the filler from working loose and being carried away by traffic.

On two other Connecticut jobs, one completed in 1937, the other in 1938, a rubber compound was used as both filler and seal. By November 1939 approximately 10 percent of the joints had been refilled with asphalt. The seals that were in place still retained their elasticity but had separated or pulled away from the concrete, allowing dirt and moisture to enter.

In Maine in June 1938, 10 expansion joints were sealed with a rubber compound consisting essentially of rubber latex, granulated cork, and a filling powder. In November 1939 all the seals were found to be in place but more than half no longer adhered to the concrete and did not appear to be watertight.

In Maryland in May, June, and July 1939 the same kind of poured rubber compound was used on three jobs to seal the top 1.5 inches above the regular bituminous fiber joint fillers. On the first job about 25 percent of the expansion joints had to be cleaned out and resealed before proper bond was secured between the seal and the concrete. When inspected late in October it was found that a great many of the joints again required resealing. On the other two jobs the expansion joints were rated as "excellent" when first completed. When inspected late in October, however, the bond between the filler and the concrete had failed on many joints to the extent that moisture could gain easy access. There were also indications on both jobs that the material had begun to lose its elasticity. These failures all occurred within less than 6 months after construction.

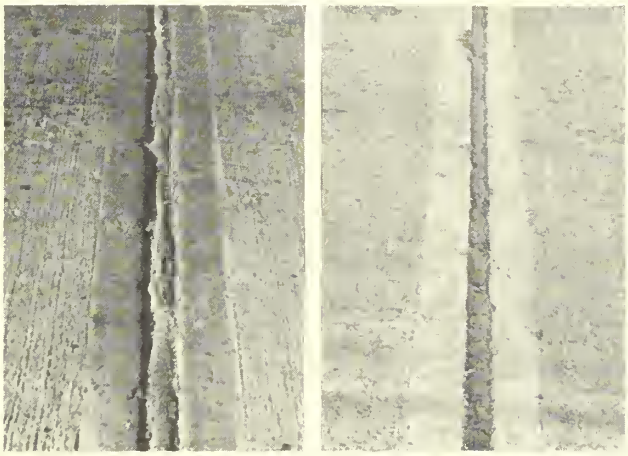
On one Massachusetts job completed in 1938 three types of poured rubber filler and seal were used. Of these, 59 joints were formed from a rubber material which was poured cold, 30 from a rubber material poured hot, and 4 were filled with a compound made up essentially of rubber latex, ground mica, and other materials. When inspected on November 17, 1939, it was found that all of the 59 cold-poured joints had lost so much of the bond with the concrete that the seal was ineffective. The hot-poured joints were tight, with good bond, and resembled in appearance an ordinary asphalt joint with considerable extrusion flattened out over the adjacent concrete. In the four poured-rubber-compound joints the material still retained its elasticity but the bond was found to be ineffective to some extent in each joint.

In another Massachusetts job completed in November 1936, all transverse joints were filled with a poured-rubber compound. The first installation failed to bond and the joints were refilled by the manufacturer in May 1937. Later that year the joints were thoroughly cleaned out by the producer of the material and completely new joints were poured. On November 20, 1939, nearly every joint was found to be cracked through the middle and had opened sufficiently to permit the free access of water and foreign materials. There was no appreciable extrusion. New joints were required.

In 1937 two poured-rubber compounds were used on a four-lane road in Massachusetts. The joints on one lane were filled with a material consisting essentially of rubber latex and a filling powder; the transverse joints of the other three lanes were filled with a material consisting of rubber latex, powdered filler, and granulated cork in the ratio of 16:4:1 by weight. The final seal was the same material without the cork. On November 22, 1939, it was found that joints of the first material had only a few cracks in the middle and that the material itself was still very elastic but the bond with the concrete was not very good. The rubber latex cork material, on the other hand, was found to have surface cracks in nearly every joint sufficient to permit the ready entrance of water and foreign materials. Adherence to the concrete was better than in joints where the first material was used. Traffic, however, had stripped the top seal to a depth of about half an inch in many places.

In 1938 the top 2 inches of the expansion joints on two Pennsylvania jobs were sealed with a rubber compound consisting essentially of rubber latex, filling powder, and granulated cork. The results were unsatisfactory even at an early date. On some of the joints the rubber compound was removed and the joints filled with bituminous material. In October 1939 the State maintenance forces sealed the remaining joints with asphalt.

In 1938 a different rubber-latex-filler compound was used on several other jobs to seal the top 2 inches of the transverse expansion joints. In October 1939 the joints on all these jobs showed evidence of functional inadequacy. The joint material was pulling out, the edges were spalling, and the material itself showed signs of rapid deterioration.

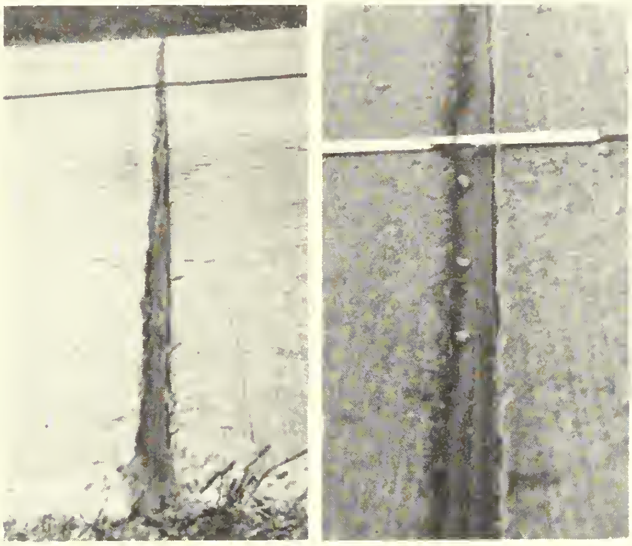


Left, Appearance of a Poured Rubber Composition Seal After 16 Months of Exposure to Weather and Traffic; Right, a Bituminous-Filled Joint in Good Condition After 2½ Years.

PREFORMED RUBBER CAPS SHOWED TENDENCY TO ROTATE AND BUCKLE

Another form of rubber seal sometimes used is a formed or manufactured strip with a hollow core. This strip is compressed during installation so that when inserted and seated in the joint it will form a tight cap or seal. It then depends on its resistance to the deformation which it has undergone by being forced into the joint opening to provide sufficient pressure against the edges of the concrete slabs to maintain a watertight joint. The early trials indicated a strong tendency for the strip either to be forced down too deeply into the joint or to rotate partially and work to the surface where it was frequently torn out by passing traffic. In later trials the slab ends have been constructed with a square shoulder or seat about one-fourth inch wide formed in the concrete about 1¾ or 2 inches below the surface of the slab. The joint opening is thus one-half inch wider at the top than at the bottom. However, considerable difficulty has been experienced in forming this seat to the required exactness. Present indications are that the tendency of the strip to rotate has not been greatly decreased. Inspections on 16 different jobs showed that very few of these installations were entirely watertight. A tendency for the strip to work up and be pulled out by the action of passing traffic has also been noted.

Experiences reported from Indiana, Mississippi, and Ohio illustrate some of the advantages and disadvantages of this type of joint. On one project in Indiana joints of this type have been in service for 4 years. When inspected late in 1939 they were in satisfactory condition and showed no serious signs of deterioration. The air space in the joint below the rubber cap was often found to be full of water, but no large amount of silt had collected in the bottom. The ability of this type of rubber cap to keep the top of the joint space closed when the pavement contracts is very promising. There seems to be no advantage in supplementing this type of joint with crack filler. There is a tendency, however, for the rubber to be pushed upward so that the wheels of traffic hit it and wear it out. This tendency is probably the result of compression caused by the closure of the joint in warm weather. It is believed that the compression causes the rubber to elongate and buckle sufficiently to push it above the surface of the pavement and expose it to wear.



Left, a Premolded Rubber Cap That Has Worked Up and Been Subjected to Hammering by Traffic; Right, a Premolded Rubber Cap That Has Rotated and no Longer Seals the Joint.

In Mississippi joints of this type were installed in November 1938 on about 1,000 feet of a 5.6-mile job. When inspected on November 9, 1939, the joint material appeared to be in good condition, but tests with a knife blade indicated insufficient pressure of the rubber against the concrete slab to exclude water at all points. In one joint approximately 9 feet of the rubber sealing strip was gone. It appears that this rubber strip can be removed without the use of tools. Breaks were also noted in the strips where they are bent 90° at the edge of the pavement to carry the strip down the side of the slab.

The use of rubber or rubber compound as a seal or joint filler on Federal-aid work in Ohio has been practically limited to a preformed extruded rubber expansion joint filler or cap as a seal. The first installation of this material was in October 1936. The joints soon proved unsatisfactory, chiefly because of a tendency to become depressed too deeply into the joint opening, or to rotate within the joint opening and permit the entrance of water and foreign materials.

Later designs provide for the formation of a shoulder on each side within the joint opening on which the rubber seal rests. This is obtained by making the lower part of the joint opening 1 inch wide and the upper part in which the rubber is inserted, 1.25 inches wide. The shoulder on each side would then be one-eighth-inch wide at the top. Considerable difficulty has, however, been encountered in forming a true and neat shoulder. An inspection late in 1939 of the the seven Federal-aid jobs on which this type of extruded rubber expansion joint was used, showed the conditions summarized in table 1.

A number of State highway departments have done considerable work toward developing satisfactory materials for sealing the tops of expansion joints. These materials are for the most part composed of a mixture of rubber latex with asphalt or tar, to which other substances such as ground cork, flake mica, or other ground or powdered substances are frequently added.

Table 1.—*Condition of extruded rubber expansion joints used on 7 jobs in Ohio*

	Job number—						
	1	2	3	4	5	6	7
Date completed	7-20-39	7-20-39	8-12-39	9-11-39	(1)	(1)	(1)
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Joints with filler too high	4	5	3	2	2	1	2
Joints with filler too low	12	14	10	8	1	1	1
Joints with filler extruded above pavement surface	2	3	1	1	0	0	0
Joints with filler cracked	1	2	1	1	0	0	0
Joints with incipient spalling of concrete	6	7	5	3	2	1	2
Joints with progressive spalling of concrete	2	3	2	1	0	0	0
Joints with filler rotated	2	2	0	0	0	0	0
Maintenance to date	None	None	None	None			

[1] Under contract; construction not yet completed when inspected.

SEALS CONTAINING BITUMINOUS MATERIALS USED WITH VARYING SUCCESS

California has used a material consisting essentially of about 70 percent SC-4 cutback and 30 percent rubber latex as both a seal and a filler. This material was apparently first used on a Federal-aid job in 1935 on about 680 linear feet of expansion joints on the heavily traveled highway between Lebec and Grapevine in Kern County. These joints, spaced at 100-foot intervals, were sealed immediately after the concrete surface was completed. Due to the dampness of the concrete, short sections of the seal were soon pulled out by traffic and had to be replaced. There have been no further failures, and no maintenance has been required to date. When inspected in October 1939 the rubber compound joints were intact and in good condition.

Later experience with this material indicates that, while superior in producing an effective seal, it must be relatively soft in order to be effective. However, when this material is soft it is also very adhesive so that foreign materials readily adhere and are kneaded into the joint under the action of traffic. In many instances sufficient extraneous materials have in this way been accumulated and worked into the joint filler to reduce seriously its ability to provide for expansion. The soft consistency of the material also permits it to flow out of the joint on sections having high superelevation. The cost is approximately double that of the standard joint fillers.

Oregon has used as a seal, with what are said to be promising results, a mixture of 30 percent rubber latex (60 percent solution) and 70 percent of 150–200 penetration asphalt. This mixture is poured hot and the seal allowed to come to within about one-fourth inch of the level of the pavement surface. The hot material is then covered with sufficient rubber grindings, of the kind and size ordinarily obtained from tire retread establishments, to complete the final seal and filling of the joint.

Massachusetts and Oklahoma have also been using to some extent somewhat similar compounds as a seal or filler. The formula used by Oklahoma is given in table 2.

Table 2.—*Formula for joint filler used by Oklahoma*

Material	Amount by—	
	Weight	Volume
	Lb.	
SC-4 oil	5.475	0.690 gal.
38 percent latex	2.431	0.299 gal.
Lime	.0553	0.703 oz.
Cresol soap solution	.0444	14.3 cc.

That at least some of these rubber materials and compounds have considerable merit is indicated by the number of instances in which their use has given satisfactory results. On the other hand a much larger number of reports of unsatisfactory results indicate that as a class these rubber compounds are still in the experimental stage. In some cases the materials or combinations of materials appear to be at fault. In other cases there is considerable evidence that the materials are capable of giving satisfactory results but more definite knowledge must be obtained as to how and under what conditions they can be successfully used. Weather conditions during installation, especially temperature and moisture, appear to affect the performance of most, if not all, of the poured types.

(*Continued from page 56*)

Local motor-vehicle imposts in Missouri are most extensively used by the larger cities. In the smaller places the greater part of the street work is accomplished by means of State highway expenditures within their coporate limits.

The relatively small increases in collections from local motor-vehicle license fees at a time of rapidly increasing motor-vehicle registrations in the State suggest possible difficulty in the collection of the local taxes and make it apparent that some more satisfactory method of obtaining an equal or larger amount of revenue is desirable.

Complete data are not available on the cost of collecting the local taxes, but there is evidence that these costs are high, and when compared with State costs for similar operations, absorb a large percentage of the total collections.

Difficulties have been encountered in the collection of the local motor-fuel taxes. Particularly in the smaller places, evasion by means of the location of gasoline stations outside city limits is frequent. This difficulty is lessened in the larger cities where the size of the cities and the number of vehicles require many refueling stations inside city limits.

The experiences of many States indicate the difficulty of completely stopping bootlegging of gasoline and other evasions of the motor-fuel tax. Within Missouri, where municipal motor-fuel taxes range from 1 cent per gallon downward, the incentive for tax evasion across city limits may not be as great as it is across State lines, where the differential in State taxes may be from 1 to 5 cents per gallon. However, the incentive does exist and it can hardly be doubted that the cities that levy local motor-fuel taxes would collect larger amounts but for evasion.

Both the local motor-vehicle license fees and the local motor-fuel taxes are duplications of State-levied taxes from which much larger amounts are obtained for the State's use. In addition to duplication of tax there is also duplication of tax collection and administration operations within the State.

Local motor-fuel taxes and motor-vehicle license fees are not used extensively in other States to finance local streets. Funds for that purpose are derived in those places from other forms of local taxation or from financial assistance provided by the county or State. The use of this form of local financing has not tended to increase in recent years, either in Missouri or in other States. Statutory restrictions and practical administrative objections have led the local units elsewhere generally to seek other methods of financing street construction and maintenance.

APPENDIX

Provision for limiting local license fees was initially made in the Missouri Revised Statutes of 1919. The law, limiting local fees to one-half of the State license fees, was continued by a 1921 law.

As the result of an initiative petition passed by the electorate of Missouri in 1924 the State license rates were increased 50 percent in 1925.[1]

The provision of the 1921 act which permitted the municipalities to levy a local license fee at one-half the State rate was replaced by the new provision limiting the local rates to one-third of the increased State rates.

The Revised Statutes of 1929 contained two conflicting provisions.[2] Sections 7761 and 7762, which were carried forward from section 5 of the 1921 act and section 15 of the 1925 act, limited "license taxes levied on motor vehicles of the State by municipal corporations" to not more than "one-third of the aggregate amount of the State registration fee * * * including the cost of plates and notarial fee." However, in section 7780 of the same statutes, which was brought forward from the 1921 sessions acts, the change from one-half to one-third was not made. Consequently there was found in this section the statement that municipalities could require (except in cities having a population of more than 75,000) the display of license plates "but such license taxes shall not exceed one-half of the registration fee provided for herein, including the cost of plate and notarial fee."

Thus, there were two different provisions in the statutes. One provision restricted the local fees to one-third of the State fees and the other permitted the towns and cities to charge license fees up to one-half of the State fees. The latter provision, contained in section 7780, should have been revised by reason of enactment of sections 7761 and 7762, but was allowed to remain in the statutes through error. An opinion by the Legal Department of the State Highway Department regarding section 7780 contained two rather lengthy supreme court decisions and stated in part: "The fact that the committee brought the said act forward and placed it in the revised statutes gave it no validity." The statement is made that the actual revision had occurred through enactment of other sections and, "The sections of the act were simply brought forward and placed in the article by the committee on revisions which was appointed to compile, arrange, and publish statutes after the adjournment of the general assembly. That committee had no legal power conferred upon it, for the legislature could not, and, indeed, did not attempt to delegate to it, any such powers."

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[1] Session Acts of 1925, sec. 15, p. 288.

[2] Ch. 41, art. 1.

The rate of one-half of the State license fees contained in section 7780 was, therefore, not permissible because of revisions of sections 7761 and 7762. As a result of the changes in the law and the conflicting provisions, considerable confusion existed as to the legal basis for establishing municipal license fees. Some cities succeeded in operating under section 7780 which permitted local rates at one-half the State rates, even though this section legally was noneffective.

At the extra session of the Missouri Legislature in 1933–34 the State license fees (established by the 1925 act) were lowered. Sections 7761, 7762, and 7769 were repealed and new sections enacted bearing the same numbers. The last paragraph in the revised section 7761 stated: "Fees charged by municipalities for said license shall not exceed the amount authorized therefor by said municipalities during the year 1933." This provision allowed those rates which had been based on the higher (1925) State rates and established by the cities prior to 1933 to remain unreduced but prohibited the municipalities from increasing the local rates in effect at that time. Since the State rates had been lowered it was therefore possible for a city rate to exceed one-third the new 1934 State rate without being in conflict with the statutes.

However, through an oversight section 7780 was not revised during the 1933–34 session. The two conflicting provisions regarding the basing of local fees on one-third and one-half of the State fees continued to cause confusion until section 7780 was finally repealed and the law clarified by the 1935 session of the legislature. A new section bearing the same number was enacted in 1935 with various changes, one of which was, "Such (municipal) license fees shall not exceed the limitations on registration fees, now or hereafter provided by law."

Consequently, the schedule of license fees shown in the second column of table 2 (p. 50) was legally in effect up to 1933 and remained in effect after 1933 for those cities which had established a local motor-vehicle license fee prior to 1933. For all local fees established after 1933 the rates were based on one-third of the State rates as provided in sections 7761, 7762, and 7780 (as revised in 1935).

The statutory development of the bases for establishing municipal motor-vehicle license fee rates in Missouri from 1921 to 1935 is shown in table I.

TABLE I.—*Statutory development of municipal motor-vehicle license fee rates in Missouri, 1921–35*

Date of law	State license fee	Municipal license fees for passenger cars—	
		Based on—	Amount
1921 (continued from 1919 act)	$5.00	One-half of State rate	$2.50
	7.00		3.50
	11.00		5.50
	15.00		7.50
	17.00		8.50
	21.00		10.50
	25.00		12.50
1925 (revised the State license rates established by 1921 act).	7.50	One-third of State rate	2.50
	10.50		3.50
	16.50		5.50
	22.50		7.50
	25.50		8.50
	31.50		10.50
	37.50		12.50
1929 Revised Statutes; secs. 7761, 7762 (carried forward from sec. 5 of 1921 act and sec. 15 of 1925 act).	7.50	One-third of State rate	2.50
	10.50		3.50
	16.50		5.50
	22.50		7.50
	25.50		8.50
	31.50		10.50
	37.50		12.50
1929 Revised Statutes; sec. 7780 (carried forward from sec. 24, par. C, of 1921 act).	7.50	One-half of State rate (This provision allowed to remain in statutes through error. Rate not permissible because of secs. 7761 and 7762.)	3.75
	10.50		5.25
	16.50		8.25
	22.50		11.25
	25.50		12.75
	31.50		15.75
	37.50		18.75
1933–34 (Revised the State license rates established by 1925 act, but did not eliminate conflicting provisions of 1929 Revised Statutes).	5.00	One-third of State rate for new local fees. Previously established rates not affected.	1.66
	8.50		2.83
	11.00		3.66
	20.00		6.66
	25.00		8.33
	31.50		10.50
	37.50		12.50
		One-half of State rate for new local fees. Previously established rates not affected. (This provision allowed to remain in statutes through error. Rate not permissible because of secs. 7761 and 7762.)	2.50
			4.25
			5.50
			10.00
			12.50
			15.75
			18.75
1935	5.00	One-third of State rate for new local fees (sec. 7780 finally revised).	1.66
	8.50		2.83
	11.00		3.66
	20.00		6.66
	25.00		8.33
	31.50		10.50
	37.50		12.50

HIGHWAY RESEARCH BOARD PROCEEDINGS NOW AVAILABLE

The Highway Research Board of the National Research Council has announced that Volume 19, Proceedings of the Highway Research Board, is now available. Copies may be obtained from the Highway Research Board, 2101 Constitution Avenue, Washington, D. C., at $2.25 each.

The volume includes papers and committee reports on the subjects of Highway Finance, Economics, Design, Roadside Development, Materials, Maintenance, Traffic and Safety, Soil Mechanics, and a symposium on Soil Stabilization Practices.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF APRIL 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 994,197	$ 990,532	11	2	1	$ 803,207	$ 803,124	9			$ 15,400	$ 15,400			4	$ 884,449
Arizona	256,288	256,244				261,773	259,569	4								400,746
Arkansas	184,062	179,124	3			823,556	819,908	4	1	5	258,589	258,055	4		9	733,909
California	1,343,969	1,339,199	10		2	1,409,636	1,239,151	6	1		25,524	25,524			11	1,653,134
Colorado	654,554	619,825	5		21	283,839	283,839	1		3	2,243	2,243			1	923,864
Connecticut	26,225	12,665				517,593	511,051	4	1	7	122,792	119,143	1			617,254
Delaware	7,839	7,839	2								96,433	96,433			16	568,269
Florida	428,094	428,094	2			196,653	192,155	2			28,967	28,967			8	1,433,663
Georgia	231,598	227,450	5		4	662,198	662,198	12	4	2	304,250	304,250	2	2	16	2,405,231
Idaho	204,578	172,785	3			215,380	214,414	2			155,347	125,721	2		11	479,788
Illinois	2,578,844	2,487,989	17	4	50	2,270,080	2,106,555	11	1	44	176,018	118,690			68	2,534,371
Indiana	829,204	821,288	3	1	80	713,070	713,070	4	2	37	232,787	232,787	1		59	1,294,068
Iowa	1,120,019	1,028,454	13		177	300,116	229,338	3		31	320,901	300,725	1		76	1,365,537
Kansas	931,952	927,574	11		10	867,127	866,649	12		2	210,757	210,757	1	2	14	1,236,441
Kentucky	605,234	594,556	8	4	9	828,511	828,511	7		5	396,055	396,055	7		13	645,065
Louisiana	493,976	493,958	4			460,321	406,826	5			627,885	570,153	11			789,393
Maine	336,835	334,299	3	2		209,733	209,733	2	1		139,758	139,758	1		5	290,561
Maryland	94,896	85,788	1		3	630,009	598,216	3	1	7	18,400	18,400			6	825,125
Massachusetts	400,519	399,288	2	2		122,047	122,014	2								2,335,881
Michigan	858,163	841,271	6	2	37	1,435,948	1,435,948	6	3	7	469,339	469,339	3		14	1,255,774
Minnesota	514,972	489,276	4	4	17	1,421,338	1,420,244	10	2		502,626	502,626	2	2	12	1,300,705
Mississippi	246,300	246,300	6			590,973	590,973	5	1		233,150	233,150	2			940,174
Missouri	450,998	449,119	3	1		1,359,458	1,359,458	8	2		854,831	577,847	1	3	1	1,815,518
Montana	850,426	850,426	9			290,905	175,823	4			18,767	16,985	1		4	609,806
Nebraska	729,939	729,340	19		38	868,292	868,292	7	2	11	142,146	142,146	2		5	725,021
Nevada	211,327	206,760		4	13	4,693	4,693				46,453	46,453			24	211,439
New Hampshire	102,433	101,921	7		1	181,249	181,212	4								392,776
New Jersey	262,682	262,682	1	1	1	743,959	743,959	3	2							1,755,390
New Mexico	123,381	122,378	3		1	263,482	263,482	2			13,292	13,292			4	620,095
New York	1,906,664	1,872,132	5	8		2,993,503	2,916,713	12	11		284,649	283,799		3	8	3,932,130
North Carolina	1,227,253	1,192,153	6	5	47	806,208	806,208	11	1	10	134,330	134,330	1	1	10	1,184,581
North Dakota	525,514	476,687	7	1		433,740	433,740	6		4	113,410	113,410	1		17	780,868
Ohio	545,780	530,780	8	1	4	1,502,813	1,431,861	8	2		999,150	983,960	5	1	8	3,365,969
Oklahoma	360,653	358,718	5		50	434,527	433,627	4	1		437,204	437,204	8		27	2,131,342
Oregon	173,428	171,720	2			318,721	218,374	3			36,103	32,000	1			536,943
Pennsylvania	1,676,971	1,465,072	2	3	2	994,225	994,225	10	1		858,000	826,000	5	1		5,068,667
Rhode Island	446,089	445,450	1	3		7,406	7,406									284,644
South Carolina	483,201	449,677	7	4	7	469,383	469,383	5	5		126,375	126,375		1	26	1,075,529
South Dakota	321,253	321,253	3	2	17	126,227	126,227	2	1		59,170	59,170	1		3	1,347,042
Tennessee	283,398	269,453		2	5	590,551	590,551	3	2							1,912,511
Texas	2,382,413	2,359,522	22	3	15	1,823,168	1,751,360	15		5	132,403	111,513	1		8	2,825,208
Utah	371,344	371,182	3		131	36,468	36,468			10	13,570	13,570			4	369,442
Vermont	32,093	27,207			7	206,402	206,402		2		2,969	2,969			1	261,133
Virginia	723,745	630,845	8	3	18	199,069	197,863	3	1		117,618	117,398	1	1	5	1,286,784
Washington	338,538	337,062	3	1	12	321,278	321,278	5	2		152,687	151,187	1	1	7	557,168
West Virginia	353,381	337,621	7		1	41,170	41,170	2		1	113,000	113,000	1		2	1,253,132
Wisconsin	887,069	883,337	9		10	1,005,060	958,936	8	3	2	320,302	291,877	1	1	10	1,359,000
Wyoming	139,774	120,314	1	1	8	84,400	84,400	1								649,412
District of Columbia						359,912	326,368	1								200,203
Hawaii	168,936	166,306	2	1	1	223,256	223,249	4			4,810	4,810			1	294,506
Puerto Rico	49,040	48,840	1			365,872	363,870	9			218,135	215,466	2			414,857
TOTALS	29,470,041	28,541,755	263	65	800	32,078,505	31,050,084	254	56	193	9,536,595	8,982,937	71	19	518	62,134,518

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF APRIL 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,810,105	$ 3,298,328	283.7	$ 4,920,367	$ 2,436,713	139.6	$ 1,749,990	$ 871,390	76.8	$ 3,509,747
Arizona	2,261,514	1,584,429	120.1	1,673,121	1,126,377	76.6	144,886	81,613	6.9	1,783,755
Arkansas	5,031,960	3,991,059	226.4	2,069,967	1,106,578	93.4	992,585	490,595	50.2	1,598,915
California	5,695,326	3,052,622	90.2	6,992,876	3,674,673	117.5	2,134,867	1,083,800	50.9	4,225,532
Colorado	3,917,643	2,130,112	89.3	1,608,496	904,689	44.6	620,375	349,643	14.3	3,297,231
Connecticut	871,263	431,585	10.0	1,529,281	761,291	16.4	1,295,899	622,750	9.3	1,361,264
Delaware	844,413	403,702	31.3	1,133,702	565,363	7.2	685,244	315,213	17.2	1,278,312
Florida	3,166,669	1,580,487	33.2	4,152,831	2,070,296	123.2	649,498	324,749	7.2	2,536,320
Georgia	4,687,725	2,204,928	238.4	5,230,205	2,615,103	286.9	2,634,711	1,317,356	78.6	7,075,403
Idaho	2,205,025	1,294,157	113.9	1,023,052	624,163	78.2	394,664	238,433	23.0	2,171,193
Illinois	8,742,598	4,320,905	195.7	7,749,772	3,872,897	136.1	2,385,100	1,171,050	85.1	5,006,733
Indiana	4,588,844	2,208,246	88.7	5,307,303	2,647,245	111.8	2,571,130	1,285,480	48.7	2,947,131
Iowa	3,999,731	1,875,169	198.2	4,583,925	2,101,692	151.8	1,883,217	887,300	56.9	2,214,537
Kansas	3,889,948	1,881,676	193.9	3,280,462	1,635,944	171.9	4,727,210	2,348,658	327.7	5,006,848
Kentucky	3,331,257	1,645,809	112.3	1,755,563	876,225	31.0	2,955,684	1,477,842	68.4	3,805,588
Louisiana	1,191,830	592,686	36.7	11,703,964	2,895,343	33.9	1,672,636	824,697	50.7	3,715,371
Maine	2,136,387	1,056,517	52.4	747,557	373,778	16.7	40,680	20,340	.6	1,308,265
Maryland	2,909,976	1,403,023	39.5	1,755,275	869,320	33.2	1,931,200	661,000	15.7	1,924,651
Massachusetts	3,134,614	1,564,618	25.1	1,791,897	892,575	15.3	777,517	387,858	6.9	3,762,699
Michigan	4,868,223	2,321,815	115.4	3,952,679	1,888,339	128.2	4,877,973	2,438,987	139.7	3,187,352
Minnesota	5,792,519	2,843,636	389.5	3,542,911	1,753,995	171.0	2,173,807	1,085,889	202.0	5,518,691
Mississippi	5,985,268	2,080,999	261.4	5,518,748	2,428,135	197.5	2,564,393	1,106,952	187.2	2,543,936
Missouri	3,962,465	1,955,764	150.8	4,238,050	2,013,934	127.8	4,337,776	1,625,702	176.4	6,179,855
Montana	3,350,311	1,891,059	196.9	2,685,271	1,521,633	151.9	1,641,954	928,918	147.9	4,674,155
Nebraska	4,893,585	2,386,774	418.0	4,400,060	2,086,615	471.5	2,998,518	1,500,168	408.8	3,406,557
Nevada	1,185,255	1,003,504	57.8	1,385,005	1,191,198	69.6	7,516	6,552		1,794,469
New Hampshire	814,656	400,642	27.3	922,218	453,607	19.8	1,107,787	437,494	22.1	997,230
New Jersey	858,420	418,696	7.1	6,254,528	3,126,314	50.2	200,130	100,065	.1	2,205,194
New Mexico	2,342,048	1,421,597	172.0	1,501,352	935,561	73.1	311,921	188,706	51.8	2,141,096
New York	8,664,045	4,165,300	167.9	12,573,797	6,159,601	178.5	2,096,540	901,420	43.8	5,100,087
North Carolina	5,747,742	2,863,592	353.3	4,638,485	2,319,032	220.5	1,601,010	800,375	80.6	2,796,078
North Dakota	296,609	157,147	41.7	1,602,970	869,760	104.4	2,885,553	1,615,637	282.8	4,458,283
Ohio	6,153,802	3,016,114	86.2	9,315,092	4,634,172	78.3	6,877,551	3,438,590	72.1	6,025,974
Oklahoma	3,155,606	1,669,753	142.8	3,128,335	1,658,920	73.8	2,220,945	1,142,271	112.4	4,680,927
Oregon	2,987,948	1,788,539	113.1	3,276,833	1,848,802	134.6	671,889	397,990	30.0	1,994,053
Pennsylvania	9,618,420	4,724,878	113.7	6,742,796	3,222,572	63.8	5,568,505	2,727,007	65.2	5,361,650
Rhode Island	672,284	325,167	7.8	1,181,937	590,000	11.6	469,511	234,224	5.0	1,055,522
South Carolina	2,715,688	1,217,808	85.1	1,727,917	818,444	105.8	948,109	457,368	84.3	2,807,189
South Dakota	3,396,897	1,855,686	330.0	3,031,640	1,726,790	346.2	1,918,680	1,090,700	291.2	3,948,312
Tennessee	4,831,038	2,337,467	104.3	3,035,002	1,517,501	62.3	1,077,856	538,928	48.8	4,744,924
Texas	14,084,456	6,923,626	731.4	10,331,172	5,105,387	520.3	1,691,659	835,774	107.8	8,581,377
Utah	2,323,405	1,611,553	98.1	868,395	625,015	51.4	375,910	256,417	40.0	1,579,355
Vermont	738,233	348,926	18.4	927,657	463,616	28.9	508,070	250,028	12.0	567,484
Virginia	2,512,645	1,243,967	82.2	2,945,515	1,407,768	67.4	381,380	184,671	11.1	2,631,303
Washington	2,267,884	1,147,462	38.5	3,849,760	2,022,086	52.1	498,935	263,200	13.4	1,656,585
West Virginia	1,963,394	1,035,382	50.1	1,736,756	865,022	51.2	900,670	446,240	16.5	2,869,785
Wisconsin	5,159,018	2,529,616	187.6	5,021,815	2,464,880	153.2	498,382	245,155	16.2	4,241,006
Wyoming	1,562,428	955,758	141.3	1,866,236	1,176,853	195.9	73,408	35,855	10.9	1,463,429
District of Columbia	326,244	161,490	2.5	468,460	233,212	3.8	393,600	196,800	3.4	456,623
Hawaii	499,394	232,151	5.1	807,876	403,916	14.5	433,940	215,300	7.7	1,602,895
Puerto Rico	707,642	351,850	14.1	1,306,457	646,785	25.3	40,640	20,065	.2	916,720
TOTALS	183,854,400	93,907,776	6,890.4	183,795,341	90,229,730	5,689.7	82,601,611	40,473,215	3,686.5	160,717,591

U. S. GOVERNMENT PRINTING OFFICE: 1940

PUBLICATIONS *of the* PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration classified according to subject and including the more important articles in PUBLIC ROADS may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF APRIL 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 929,839	$ 358,494	42.2	$ 627,812	$ 312,930	24.8	$ 450,750	$ 224,200	27.7	$ 642,327
Arizona	282,201	201,025	33.4	201,552	123,926	23.1				429,402
Arkansas	977,339	828,905	95.9	119,046	78,669	8.7	33,318	16,659	8.7	347,898
California	848,184	466,095	36.1	539,553	297,799	26.9	281,708	147,500	6.1	1,020,538
Colorado	963,432	495,540	33.4	206,730	80,888	5.1				281,949
Connecticut	172,310	72,417	2.9	240,804	116,483	2.3	137,486	61,921	2.3	205,673
Delaware	84,115	39,067	17.5	69,537	34,768	7.8				309,475
Florida	885,834	438,113	31.3	29,916	14,958	.6	101,240	50,620	1.3	563,391
Georgia	363,706	172,132	44.2	207,103	103,551	20.9	441,959	220,980	41.7	1,273,841
Idaho	456,251	247,154	45.1	128,663	78,481	12.8				295,190
Illinois	1,450,364	647,630	87.4	1,406,585	701,118	79.7	593,100	281,550	11.0	643,981
Indiana	896,572	423,398	77.7	352,270	174,931	23.4	214,670	107,300	14.9	947,730
Iowa	372,298	178,473	132.6	1,674,040	796,305	244.7	749,152	352,225	158.6	733,037
Kansas	104,423	52,195	46.4	606,374	303,187	8.9	91,949	45,974	14.2	1,568,855
Kentucky	1,111,408	325,230	71.4	598,561	202,184	41.3	533,452	173,435	59.9	420,773
Louisiana	891,686	431,549	76.2	305,299	152,595	24.7	25,496	12,748	4.4	451,252
Maine	437,626	210,484	25.4	81,124	39,773	4.0				141,656
Maryland	281,641	137,157	19.2	112,996	56,498	4.4	16,000	8,000	2.2	457,194
Massachusetts	373,212	185,203	9.2	443,272	219,589	10.1	116,250	57,255	3.0	533,816
Michigan	1,305,971	635,144	113.8	995,439	501,109	50.6	951,185	469,348	86.0	584,828
Minnesota	946,363	460,583	122.3	355,950	146,104	25.8	224,893	112,447	40.8	1,343,910
Mississippi	359,900	179,919	40.7	1,063,962	520,196	52.6				713,924
Missouri	988,883	486,972	145.6	568,222	278,397	48.9	285,692	112,671	52.3	837,137
Montana	809,775	456,469	71.6	403,474	226,230	44.8	270,760	152,983	34.0	789,246
Nebraska	1,062,182	505,194	213.9	439,625	219,654	62.9	375,073	187,536	52.7	408,163
Nevada	164,206	134,299	25.0	197,591	169,920	25.5	22,792	19,870	4.8	208,472
New Hampshire	58,755	29,144	2.3	140,209	66,301	4.2				197,644
New Jersey	329,558	161,550	12.1	415,200	207,600	15.3	170,240	85,120	3.0	540,280
New Mexico	469,280	287,227	42.1	367,851	229,448	26.9				262,447
New York	1,822,988	877,207	91.3	1,688,675	810,474	45.2	551,441	251,871	19.1	715,364
North Carolina	1,069,398	531,728	106.2	730,323	366,798	66.9	275,860	129,600	22.3	370,754
North Dakota	115,841	61,370	8.3	62,121	34,895		156,003	83,616	3.8	1,009,589
Ohio	700,034	347,437	41.8	1,064,442	538,996	45.2	2,146,120	1,057,560	61.4	888,539
Oklahoma	318,926	163,505	16.3	730,196	387,787	53.6	160,900	84,987	14.1	1,039,633
Oregon	628,592	356,472	71.7	351,387	192,987	37.8	118,175	67,540	23.7	367,956
Pennsylvania	2,018,709	969,589	117.1	1,369,144	678,467	46.8	1,256,713	627,594	30.8	242,241
Rhode Island	93,827	46,890	2.2	230,055	115,001	3.6				95,049
South Carolina	578,846	234,056	56.9	440,040	163,195	38.1	238,054	94,629	31.2	296,605
South Dakota	16,229	8,937	4.1	14,680	9,712					1,280,913
Tennessee	818,717	351,682	31.7	251,454	125,727	8.6	42,142	21,071	3.0	1,000,026
Texas	2,629,740	1,294,611	291.3	1,165,718	570,898	151.2	280,190	126,610	50.1	1,266,363
Utah	295,257	167,363	43.4	52,625	35,930	3.3				297,598
Vermont	138,763	63,935	5.6	164,292	50,336	6.3	249,200	77,029	8.7	58,289
Virginia	648,377	314,151	65.5	497,694	222,770	22.2	67,100	33,550	9.5	384,281
Washington	574,915	298,438	43.1	326,636	171,357	20.7	100,518	53,354	12.0	380,818
West Virginia	145,150	72,575	8.3	361,065	180,007	20.3	113,508	56,754	5.7	445,914
Wisconsin	882,197	438,333	34.5	321,053	160,190	4.7	397,600	191,780	5.3	821,408
Wyoming	470,702	286,620	26.0	383,424	229,898	38.9				189,322
District of Columbia	84,004	41,650	1.0	41,692	20,346	.5				84,254
Hawaii	193,033	96,211	6.2	273,208	137,108	8.6				159,806
Puerto Rico	30,220	14,440	2.2	244,135	119,025	12.9	128,458	62,985	3.2	73,178
TOTALS	32,651,779	16,283,962	2,791.6	23,662,819	11,775,496	1,567.1	12,369,147	5,920,872	933.5	28,621,929

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 21, NO. 4 JUNE 1940

PERFORMING BENDING TESTS ON BITUMINOUS MIXTURES

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLICATIONS
JUL 8 1940

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

FEDERAL WORKS AGENCY

PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 4 June 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CHEMICAL TREATMENT OF CHERT-GRAVELS FOR USE IN BASE-COURSE CONSTRUCTION

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **E. A. WILLIS**, Associate Highway Engineer, and **P. C. SMITH**, Junior Highway Engineer.

THIS REPORT is the fourth in a series describing investigations of materials for base-course construction. Previous reports have described laboratory and circular track tests on sand-clay and sand-clay-gravel materials and on nonplastic materials with admixtures of water-retentive chemicals.

The present report is the result of investigations in which three chert-gravels from Alabama were tested in the outdoor circular track. The chert-gravels tested were representative of a class of local materials found to such an extent in the Southeastern United States as to be important in road construction. They consist of mixtures of coarse chert particles and fine material made up of dust of fracture and clay. As a class they have given satisfactory service when used as surface courses, but have caused failures in numerous instances when used as base courses because of excessive amounts of the active binder which they usually contain.

Sand and granulated slag were used to reduce the plasticity index of the plastic chert-gravel that was investigated. Due to the presence of finely divided silica in the chert it was felt that the addition of lime might reduce the activity of the natural binder and form a cementing agent through puzzolanic action. Consequently, an admixture of hydrated lime was used in three sections to determine its effect on the behavior of the chert-gravels as base courses.

The circular track used in these investigations was the same as was used in the studies of water-retentive chemicals as admixtures with nonplastic road-building materials which have been reported previously.[1] The tire equipment was size 30 by 5, of the high-pressure type, requiring an inflation pressure of 80 pounds per square inch. The load imposed by each wheel was 800 pounds until near the end of the test when it was increased to 1,000 pounds.

Distributed traffic, which was used for compacting the base course and the surface treatment, was obtained by gradually shifting the rotating beam longitudinally with respect to its axis of rotation. Concentrated traffic, which was used after the surface treatment had been constructed, was obtained by locking the sliding pivot of the beam in such a position that the wheels pursued two concentric circular courses whose center lines were about 2½ inches on either side of the center line of the test sections.

COMPOSITION AND TESTING OF TRACK SECTIONS DISCUSSED

Six sections were tested in this investigation. Each section was 18 inches wide, 6 inches deep, and approximately 6.3 feet long.

Three chert-gravel materials were used in the track sections. Chert A was a mixture of materials from two pits in Alabama having similar test constants. This mixture had a liquid limit of 46 and a plasticity index of 20. Chert B was from a single pit in Alabama and had a liquid limit of 25 and a plasticity index of 2. Chert C was a composite of 11 samples which had been taken from chert base courses in Alabama at locations where base failures had occurred.

The compositions of the six sections in the track are given in table 1. Section 1 consisted of 45 percent of chert A and 55 percent of Potomac River sand; section 2 consisted of 43 percent of chert A, 52 percent of Potomac River sand, and 5 percent by weight of hydrated lime; section 3 consisted of 35 percent of chert A and 65 percent of granulated slag; section 4 consisted entirely of chert B; section 5 consisted of 95 percent of chert B and 5 percent of hydrated lime; and section 6 consisted of 95 percent of chert C with a 5 percent hydrated lime admixture. The quantity of sand and granulated slag used in sections 1 and 3 was that required to reduce the plasticity index of chert A from 20 to approximately 6. Section 2 was identical with section 1 except for the admixture of 5 percent of hydrated lime.

The gradings and soil constants of the mixtures used are given in table 2. The effect of the hydrated lime in increasing the liquid limit and reducing the plasticity index can be seen by comparing the analyses of section 1 with section 2 and section 4 with section 5.

TABLE 1.—*Composition [1] of sections of test track*

Section No.	1	2	3	4	5	6
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Chert A	45	43	35			
Chert B				100	95	
Chert C						95
Potomac River sand	55	52				
Granulated slag			65			
Hydrated lime		5			5	5

[1] Percentage based on dry weight.

TABLE 2.—*Gradings and soil constants of mixtures used in track*

Section No.	1	2	3	4	5	6
Grading:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Passing 1-inch sieve	100	100	100	100	99	100
Passing ¾-inch sieve	99	99	100	97	92	93
Passing ⅜-inch sieve	89	90	92	85	79	75
Passing No. 4 sieve	80	81	87	70	63	60
Passing No. 10 sieve	65	64	60	55	46	44
Passing No. 40 sieve	44	37	35	44	33	29
Passing No. 200 sieve	19	12	19	36	26	15
Passing 0.005 mm	7	5	6	11	11	9
Dust ratio [1]	43	32	54	82	79	52
Tests on material passing No. 40 sieve:						
Liquid limit	22	28	24	25	36	34
Plasticity index	8	0	5	2	0	0

[1] Dust ratio $=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

---

[1] Studies of Water-Retentive Chemicals as Admixtures with Nonplastic Road-Building Materials, by E. A. Willis and C. A. Carpenter. PUBLIC ROADS, vol. 20, No. 9, November 1939.

In constructing the test sections sufficient water was added to the aggregates to bring the fraction passing the No. 4 sieve to its optimum moisture content as previously determined by the A. A. S. H. O. standard compaction test, with a slight excess for wetting the coarse aggregates.

The moisture contents of all sections immediately after being placed in the track and at the time of failure or end of test are shown in table 3, together with the optimum moisture contents for the fraction of the materials passing the No. 4 sieve.

TABLE 3.—*Moisture contents immediately after construction and at the end of test, and optimum moisture contents of the fraction passing the No. 4 sieve*

Section No.	Optimum moisture content of fraction passing No. 4 sieve [1]	Moisture content of sections after placing [2]	Moisture content of sections at time of failure or end of test [2]
	Percent	*Percent*	*Percent*
1	11.6	10.4	9.9
2	14.0	11.2	14.6
3	14.6	10.0	12.3
4	16.1	15.2	15.1
5	19.6	16.0	19.0
6	15.8	10.0	16.7

[1] Based on the dry weight of the portion of the aggregate passing the No. 4 sieve.
[2] Based on the dry weight of the total material.

The procedure for preparing the materials for the track tests, constructing the test sections, and surface-treating them, was as follows:

1. The aggregates were proportioned by weight from the stock materials and were thoroughly mixed before any water was added.
2. Hydrated lime was added to the materials for sections 2, 5, and 6 and thoroughly incorporated in the dry mixing process.
3. Water was added and mixing continued to distribute the moisture.
4. The moistened mixtures were then placed in the trough of the track in two approximately equal layers, each layer being compacted with pneumatic-tired traffic uniformly distributed over the surface.
5. Compaction with distributed traffic was continued on the top layer for 20,000 wheel-trips, at which time no further subsidence was noted and all sections were in suitable condition for testing.
6. The sections were trimmed smooth.
7. A prime coat consisting of 0.3 gallon per square yard of light tar was applied and allowed to cure.
8. A surface treatment consisting of 0.4 gallon of hot tar and a cover of 50 pounds per square yard of ¾-inch-maximum-size stone was constructed.
9. The treated surface was consolidated by an additional 20,000 wheel-trips distributed over the surface. The surface was well sealed by then and showed no movement.

RATING OF SECTIONS BASED ON APPEARANCE AND AMOUNT OF DISPLACEMENT

The behavior of the materials under test was judged on the basis of the appearance of the sections at various stages of the tests supplemented by measurements of vertical displacements of the surface. Previous reports have described the transverse [2] and longitudinal [3] profilometers with which the measurements were made.

By means of a planimeter, the area between the initial and each succeeding transverse profile made at that station was measured. That area divided by the width of the track (18 inches) gave the vertical displacement, and the average for the two stations on the section gave the average vertical displacement for that section.

The area between the initial and each succeeding longitudinal profile made in that wheel lane was measured for each section and the area of vertical displacement determined. That area divided by the length of the wheel lane gave the depth of rutting and the average for the two wheel lanes gave the average depth of rutting for the section.

An average vertical displacement of about 0.25 inch, measured after the sections had been surface treated and subjected to the action of concentrated traffic, was observed to be sufficient to cause noticeable damage to the bituminous surface. This is in agreement with conclusions reached in previous investigations using the same apparatus. The amount of rutting measured by the longitudinal profilometer averaged approximately 0.5 inch at the same time the average vertical displacement was 0.25 inch. Since the average vertical displacement depends upon the width of the track, a comparison of average vertical displacement with depth of rutting is only valid for a track having a width of 18 inches.

Changes in behavior of the various sections under altered test conditions are clearly shown by abrupt changes in the slopes of the displacement curves, figure 1.

The schedule of traffic applications and changes in water elevation, with notations on the behavior of the five test sections, are given in table 4. Initial profile measurements were taken at 40,000 wheel-trips when concentrated traffic was started. The average vertical displacements as measured by the transverse profilometer and the average depth of ruts as measured by the longitudinal profilometer subsequent to that time are shown in figure 1.

All sections compacted well and showed little movement under distributed traffic with no water in the sub-base (0 to 40,000 wheel-trips).

Water was admitted and the level raised to one-half inch above the top of the subbase at 40,000 wheel-trips. The sections were allowed to absorb water overnight. Movement was noticed in the base course of section 4 (chert B) as soon as testing with concentrated traffic was started. At 40,572 wheel-trips, or only 572 wheel-trips after water was admitted to the sub-base, this section had failed completely and was so rough that it had to be removed and replaced with a dummy section before testing could be continued.

Profiles of section 4 after it had failed showed an average vertical displacement of 0.78 inch and an average depth of rut of 0.93 inch. The condition of the section at 40,572 wheel-trips is shown in figure 2.

Testing with concentrated traffic was resumed at 44,972 wheel-trips, 4,400 wheel-trips of distributed traffic having been applied to compact the surface treatment on the dummy section which replaced section 4.

Movement was noticed in section 1, the sand-chert A mixture, soon after testing was resumed. This continued throughout the testing period from 44,972 wheel-trips to 84,400 wheel-trips with the water level one-half inch above the top of the sub-base. The rutted condition of section 1, caused by lack of stability in the base course, is shown in figure 3.

---

[2] Circular Track Tests on Low-Cost Bituminous Mixtures, by C. A. Carpenter and J. F. Goode, PUBLIC ROADS, June 1936.
[3] A Study of Sand-Clay-Gravel Materials for Base Course Construction, by C. A. Carpenter and E. A. Willis, PUBLIC ROADS, March 1939.

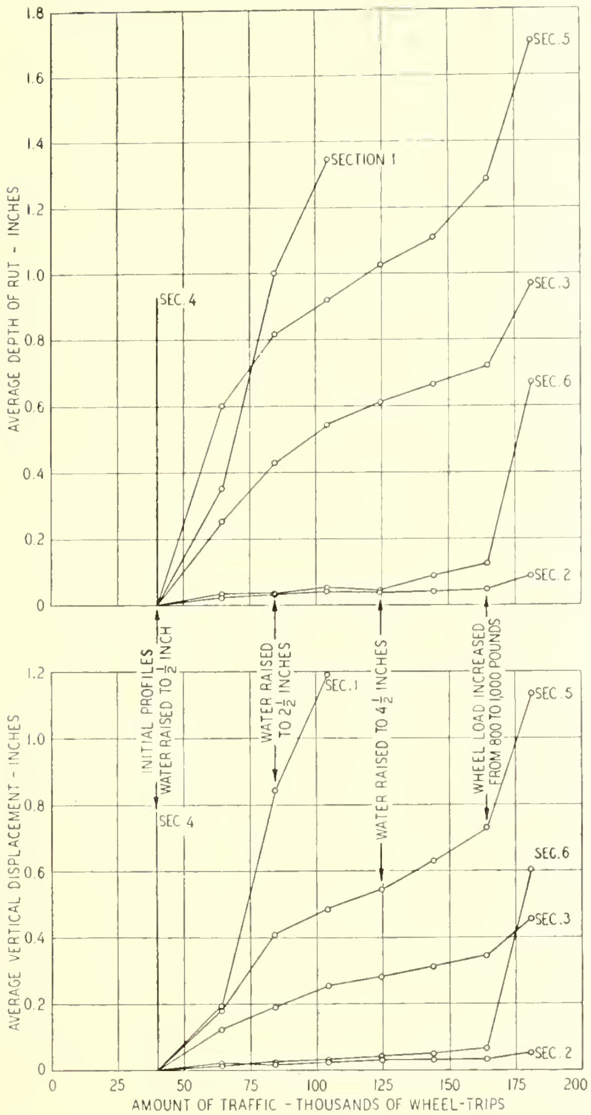

Figure 1.—Rate of Surface Displacement of Track Sections Under Traffic.

Section 5, constructed of chert B with a 5-percent admixture of hydrated lime, was slightly unstable when tested with the water at the ½-inch level. Wheel tracks were plainly visible on the surface and average vertical displacement exceeded 0.4 inch.

Sections 2, 3, and 6 remained in good condition during this period of testing, although the rate of average vertical displacement for section 3, the chert A-slag section, indicated that continued traffic with water at the ½-inch level would ultimately have produced failure.

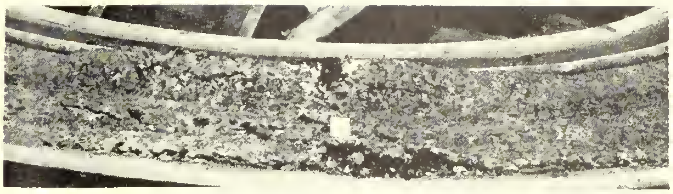

Figure 2.—Appearance of Section 4 at 40,572 Wheel-Trips, or 572 Wheel-Trips After Start of Testing With Concentrated Traffic.

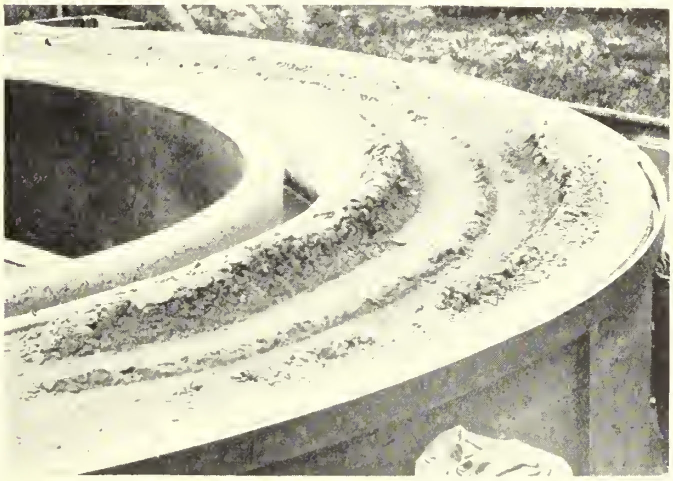

Figure 3.—Appearance of Section 1 at 84,400 Wheel-Trips, After Testing With Water ½ Inch Above the Top of the Sub-Base.

ONLY ONE SECTION REMAINED IN GOOD CONDITION THROUGHOUT ENTIRE TEST

Water was raised to 2½ inches above the top of the sub-base at 84,400 wheel-trips and testing with concentrated traffic was continued. Under these test conditions section 1 failed completely; sections 3 and 5 were slightly unstable with increasing amounts of vertical displacement; and sections 2 and 6 remained in good condition. Section 1 was removed at 124,400 wheel-trips and replaced with a dummy section.

Testing was then continued with the water raised to 4½ inches above the top of the sub-base. Wheel loads were increased from 800 to 1,000 pounds at 164,400 wheel-trips.

Table 4.—*Schedule of operations and behavior of test sections*

Operation	Traffic	Water level above top of sub-base	Behavior					
			Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5	Sec. 6
	Wheel-trips	*Inches*						
Placing and compacting	0-20, 000	[1] 0	Good	Good	Good	Good	Good	Good.
Compacting treated surface	20, 000-40, 000	[1] 0	do	do	do	do	do	Do.
Testing with concentrated traffic	40, 000-40, 572	½	do	do	do	Failed	do	Do.
Compacting treated surface on dummy sec. 4 with distributed traffic.	40, 572-44, 972	[1] 0	do	do	do		do	Do.
Testing with concentrated traffic	44, 972-84, 400	½	Unstable	do	do		Slightly unstable	Do.
Do	84, 400-124, 400	2½	Failed	do	Slightly unstable		do	Do.
Do	[2] 124, 400-180, 720	4½		do	Unstable		Failed	Slightly unstable.

---

[1] No water in sub-base.

[2] Wheel loads increased from 800 pounds to 1,000 pounds at 164,400 wheel-trips.

[3] A soft spot developed in sec. 6 during the last phase of testing which rapidly became deeper. The remainder of the section remained good. See figure 4.

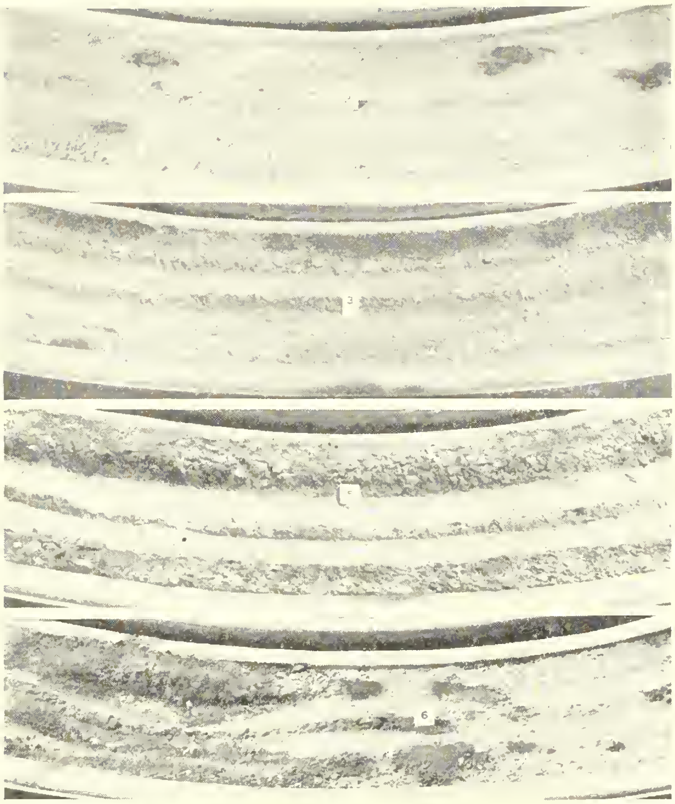

FIGURE 4.—APPEARANCE OF SECTIONS 2, 3, 5, AND 6 AT THE END OF TEST. NOTE THE SOFT SPOT THAT DEVELOPED IN SECTION 6.

At the end of this very severe treatment, section 5 had failed completely and section 3 was unstable and in very poor condition. Figure 4 shows the condition of these two sections at 180,720 wheel-trips, when testing was discontinued.

A soft spot developed in section 6, composed of chert C with 5 percent hydrated lime, at about 160,000 wheel-trips. Mud and water began to work through the surface treatment but excessive displacements were not observed. However, when the wheel loads were increased at 164,400 wheel-trips a localized failure developed at this spot on section 6. The rest of the section remained in good condition throughout the test.

The appearance of section 6 at the end of testing is shown in figure 4. The soft spot can be seen at the left.

Section 2, composed of sand, chert A, and 5 percent hydrated lime, remained in good condition at all times during the testing. The average vertical displacement on this section at the end of the test was only 0.05 inch, and average depth of ruts at the same time about 0.09 inch. The condition of section 2 at the conclusion of testing is shown in figure 4.

The density of each track section was measured at the time of failure or at the end of the testing period. The results of these determinations are shown in table 5, together with the densities obtained in the standard compaction test performed on the fraction of the material passing the No. 4 sieve.

Laboratory compaction tests were also made on the materials as tested in the track. The procedure for making these tests was similar to the standard test, except that a mold having a capacity of one-fourth cubic foot was used and the soil was compacted in three layers with 100 blows of the standard rammer per layer. The results of these tests are also shown in table 5.

TABLE 5.—*Comparison of densities obtained by laboratory compaction tests and by testing in the circular track*

Method of compaction	Section No.	Water content based on dry weight	Composition by volume		
			Water	Aggregate	Air voids
		Percent	*Percent*	*Percent*	*Percent*
Standard compaction test on fraction passing No. 4 sieve	1	11.6	23.0	72.2	4.8
	2	14.0	25.9	67.1	7.0
	3	14.6	28.2	70.2	1.6
	4	16.1	27.6	62.3	10.1
	5	19.6	31.1	57.8	11.1
	6	15.8	28.1	64.6	7.3
Compaction test on entire sample in ¼ cubic foot mold	1	13.3	26.3	71.9	1.8
	2	15.0	29.2	70.8	0
	3	14.1	26.9	69.3	3.8
	4	14.0	24.9	64.7	10.4
	5	18.0	31.0	62.7	6.3
	6	13.8	26.2	69.1	4.7
Samples cut from track at end of test or time of failure	1	9.9	20.2	74.0	5.8
	2	14.6	26.0	64.7	9.3
	3	12.3	24.0	71.0	5.0
	4	15.1	26.5	63.9	9.6
	5	19.0	32.7	62.6	4.7
	6	16.7	29.0	63.2	7.8

The addition of 5 percent hydrated lime reduced the densities of both the sand-chert A mixture and chert B in the track sections as well as in the laboratory compaction tests. Thus the amount of aggregate solids by volume for the section 1 materials in the standard compaction test was 72.2 percent; in the compaction test in the ¼ cubic foot mold, 71.9 percent; and in the track, 74.0 percent. Corresponding aggregate volumes for the section 2 materials, which were similar to section 1 except for the addition of 5 percent hydrated lime were, respectively, 67.1 percent, 70.8 percent, and 64.7 percent. Similarly, the densities of the section 5 materials, chert B with 5 percent hydrated lime, were consistently lower than the corresponding densities of the section 4 material, which was chert B without admixture.

No consistent relationship was found between densities measured by either of the laboratory compaction methods and those obtained in the track. The closest agreement was between the track densities and densities obtained by the standard compaction test.

In sections 1, 3, and 4, where hydrated lime was not used, the track densities were higher than those obtained in the standard compaction test. In sections 2 and 6, where lime was used, the reverse was true. In section 5, which also contained lime, the track densities were the higher.

SUMMARY

The grading curves for the six combinations of materials tested are shown in figure 5. The shaded band in this figure is drawn to include the A. A. S. H. O. specification requirements for coarse-graded-type aggregate base-course materials having a maximum size of 1 inch. The curve for the section 6 material is the only one which falls entirely within the specification limits. These specifications further stipulate that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 6. All mixtures tested in the track had plasticity indexes of less than 6 except that used in section 1. The liquid limits of the materials used in sections 2, 5, and 6 exceeded 25 (see table 1). All three of these sections,

(Continued on page 80)

BENDING TESTS ON BITUMINOUS MIXTURES

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **W. O'B. HILLMAN**, Assistant Highway Engineer

BENDING TESTS on bituminous paving mixtures have been made for some time. In 1923 and 1924 the Public Roads Administration made a series of such tests, the results of which were not published, which seemed to indicate some relationship between stiffness of the mixture and cracking of the pavement in service. Bituminous surfaces that had cracked in service were generally found to consist of stiffer mixes, as determined by bending tests, than those which had not cracked. These tests were made at 32° F. and 77° F. using a small universal testing machine to apply a center load on a 2-inch by 2-inch beam with a span of 10 inches.

Recently Professor Lloyd F. Rader,[1] [2] [3] [4] of Brooklyn Polytechnic Institute, made bending tests upon sheet asphalt mixtures at temperatures as low as −70° F., cooling the specimens with dry ice. He used beams 2 inches wide and 1½ inches deep with a span of 6 inches. A center load was applied with a testing machine. Both laboratory prepared specimens and specimens cut from pavements were tested. Rader studied the effect of temperature, degree of compaction, type of asphalt, and percentage of asphalt, upon the test results. At the lowest temperatures straight load-deflection curves were obtained but at higher temperatures they were considerably curved. The modulus of rupture and the modulus of elasticity were calculated. On the field specimens Rader found that in general the uncracked pavements had higher moduli of rupture and lower moduli of elasticity than the cracked pavements.

Raschig and Doyle [5] [6] have also made bending tests on laboratory prepared specimens and specimens taken from pavements. These beam specimens were 1.5 inches square, tested as cantilevers with lengths of 4 inches. Tests were made at −5, 25, and 38° F. and the modulus of elasticity and modulus of rupture were calculated.

SPECIAL BEAM TESTING MACHINE DESIGNED AND BUILT

These previous investigations have indicated the possible value of the flexure test as a means for studying the essential characteristics of bituminous paving mixtures and the reasons for the variable behavior of bituminous pavements. This investigation was undertaken primarily to study the effect of different variables upon the test results obtained with laboratory specimens although, as a matter of interest, some data from tests on samples taken from pavements are included in the report.

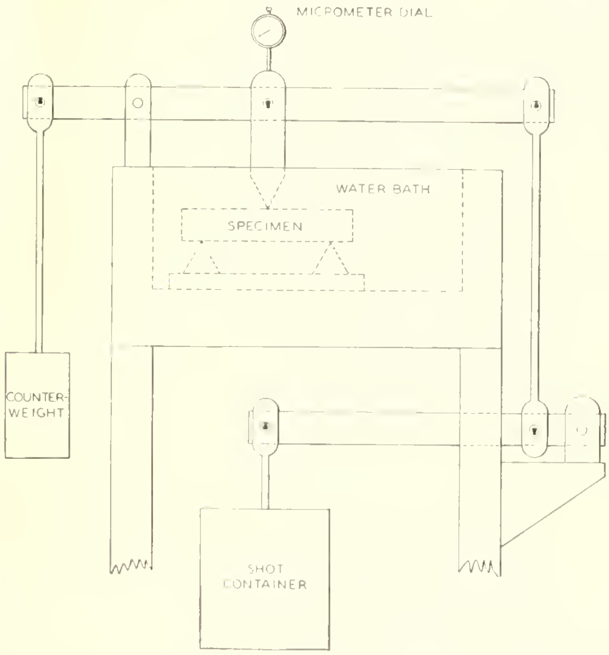

FIGURE 1.—DIAGRAMMATIC SKETCH OF BENDING APPARATUS.

Bituminous paving mixtures are plastics and therefore the results of bending tests on them depend to a great degree on the rate at which loads are applied. Since a testing machine that would apply the load smoothly and at a uniform rate, and still be sensitive in the low range of load necessary, was not available, it was necessary to design and build a special machine for making these tests. A sketch of this machine is given in figure 1 and a general view on the cover page of this issue.

It consists of two beam supports which can be adjusted for spans of 6, 8, and 10 inches, and a multiplying lever arrangement for applying a load at the center by means of lead shot. A water bath is used so that the specimen can be maintained at a definite temperature during the testing operation. Deflections are measured with an Ames dial which measures the travel of the top lever. Specimens as large as 3 inches by 3 inches can be tested. The rate of load application can be varied from about 75 pounds per minute to about 500 pounds per minute.

A limited number of tests have been made so far primarily to try out the apparatus, determine the effect of controllable variables upon the test results, and decide upon a standard method of test. Laboratory specimens were made of Potomac River sand, limestone dust, and 50-penetration Mexican asphalt. In order to reduce as much as possible the hardening of the asphalt during the fabrication of the specimens, only sufficient mixture to make one beam was prepared at a time. Beams were formed in a 2-inch by 11-inch mold by direct compression of 3,000 pounds per square inch

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[1] Investigations of the Physical Properties of Asphaltic Mixtures at Low Temperatures, Proceedings of the Association of Asphalt Paving Technologists, January 1935.

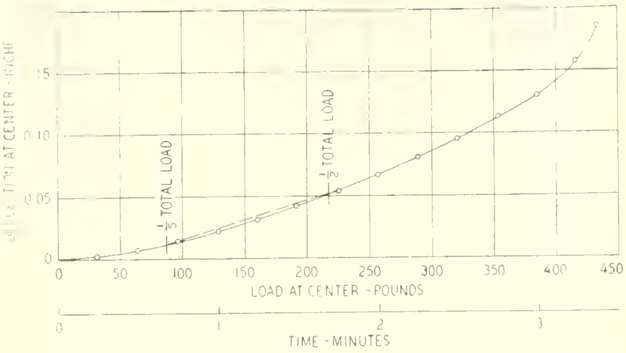
[2] Investigations of the Physical Properties of Asphaltic Mixtures at Low Temperatures, Proceedings of the American Society for Testing Materials, 1935.

[3] Correlation of Low Temperature Tests with Resistance to Cracking of Sheet Asphalt Pavements. Proceedings of the Association of Asphalt Paving Technologists, January 1936.

[4] Report on Further Research Work on Correlation of Low Temperature Tests with Resistance to Cracking of Sheet Asphalt Pavements. Proceedings of the Association of Asphalt Paving Technologists, January 1937.

[5] Some Recent Research on Asphalt Pavement. Proceedings of the Association of Asphalt Paving Technologists, January 1937.

[6] An Extension of Asphalt Research as Reported in the 1937 Proceedings. Proceedings of the Association of Asphalt Paving Technologists, December 1937.



SPECIMEN 2E. TEST TEMP 39°F. SPAN 8 INCHES. DEPTH 1.99 INCHES. WIDTH 2.07 INCHES
BREAKING LOAD 433 POUNDS. RATE OF DEFLECTION 0.00031 INCHES PER POUND

Figure 2.—Typical Load-Deflection Curve.

using two opposed plungers. The thickness was controlled by the amount of material placed in the mold.

On the day following molding of the specimens they were tested for specific gravity and then brought to test temperature by immersion in a water bath for 1½ hours. They were then placed in the test apparatus with plates ½ inch wide between the beam and the knife edges to keep the knife edges from indenting the beams. Loading was started and the Ames dial read at convenient time intervals until the beam broke. As the shot flows at a uniform rate, the load is proportional to the time and the load-deflection curves were plotted as in figure 2.

It is customary in reporting the results of bending tests to use the modulus of rupture as a measure of the strength of the material and the modulus of elasticity as a measure of the stiffness. The modulus of rupture was calculated from the formula

$$S=\frac{3Pl}{2bd^2}$$

where S = the modulus of rupture in pounds per square inch;
P = the breaking load;
l = the span in inches;
b = the width of the beam; and
d = the depth of the beam.

None of the load-deflection curves was straight and it was necessary to calculate a secant modulus of elasticity. Because of difficulty in correctly ascertaining the initial dial reading, only the portion of the curve between one-fifth and one-half the breaking load was used. The formula for modulus of elasticity then became

$$E=\frac{l^3}{4bd^3R}$$

where E = the modulus of elasticity in pounds per square inch;
R = the average rate of deflection in inches per pound of load; and
l, b, and d are the same as in the formula for modulus of rupture.

RATE OF LOADING AFFECTED MODULI OF RUPTURE AND ELASTICITY

Tests were first made to determine the effect of changing the rate of loading upon the test results. Identical beams were tested at 39° F. using three different rates of loading. The results, plotted in figure 3, show that increasing the rate of loading increases the modulus of rupture and the modulus of elasticity.

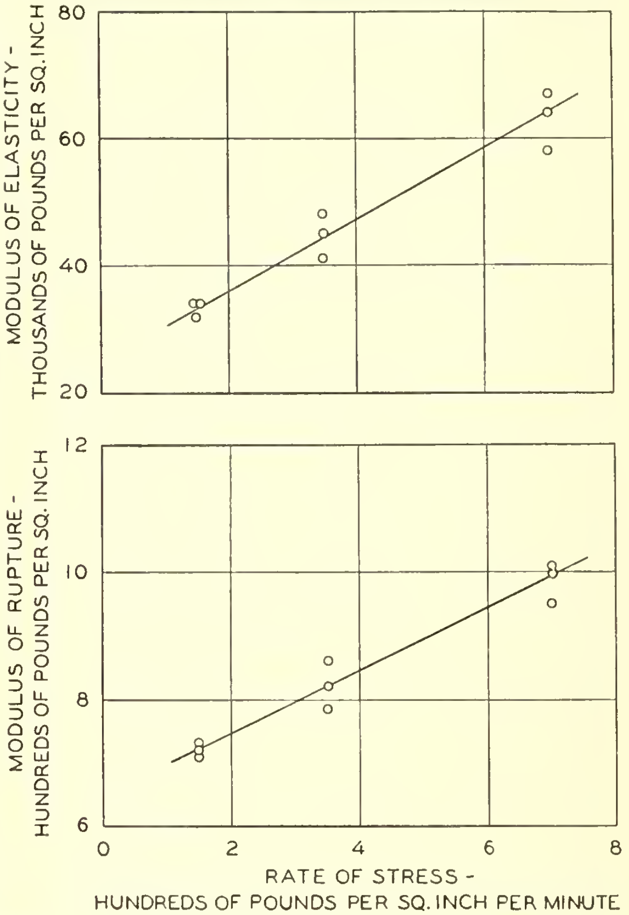


Figure 3.—Effect of Changing Rate of Loading Upon the Test Results.

In connection with the tests to determine the effect of rate of loading, a few beams were tested with constant loads. Time-deflection curves for these beams are shown in figure 4, together with the curve for a beam tested in the usual manner. It required almost 4 minutes and a load of 485 pounds to break the beam when the load increased at the rate of 125 pounds per minute. However, a constant load of 353 pounds, less than ¾ of the increasing load, broke the beam in less than 3 minutes. Loads of 295 and 235 pounds broke the beams in about 5 and 13 minutes, respectively.

Tests were made to determine the validity of the formulas for modulus of rupture and modulus of elasticity when applied to bituminous beams by testing beams of the same composition but of various dimensions and with various spans. Because of the great difference in test results obtained with different rates of loading, it was necessary to assume that the modulus of rupture formula was applicable and to vary the rate of loading directly as the width of the beam, directly as the square of the depth of the beam, and inversely as the test span. The rate of stress, calculated from the modulus of rupture formula, would thus be a constant.

Specimens 1, 1½, and 2 inches deep were made. For testing beams of different widths these were turned on edge to give beams 2, 1½, and 1 inch wide and 2 inches deep. These were tested on a span of 8 inches, the loads being applied at the rate of 120 pounds per inch width per minute. The results, given in table 1, show that beams of different widths but of the same depth when tested on the same span have practically the same modulus of rupture and modulus of elasticity.

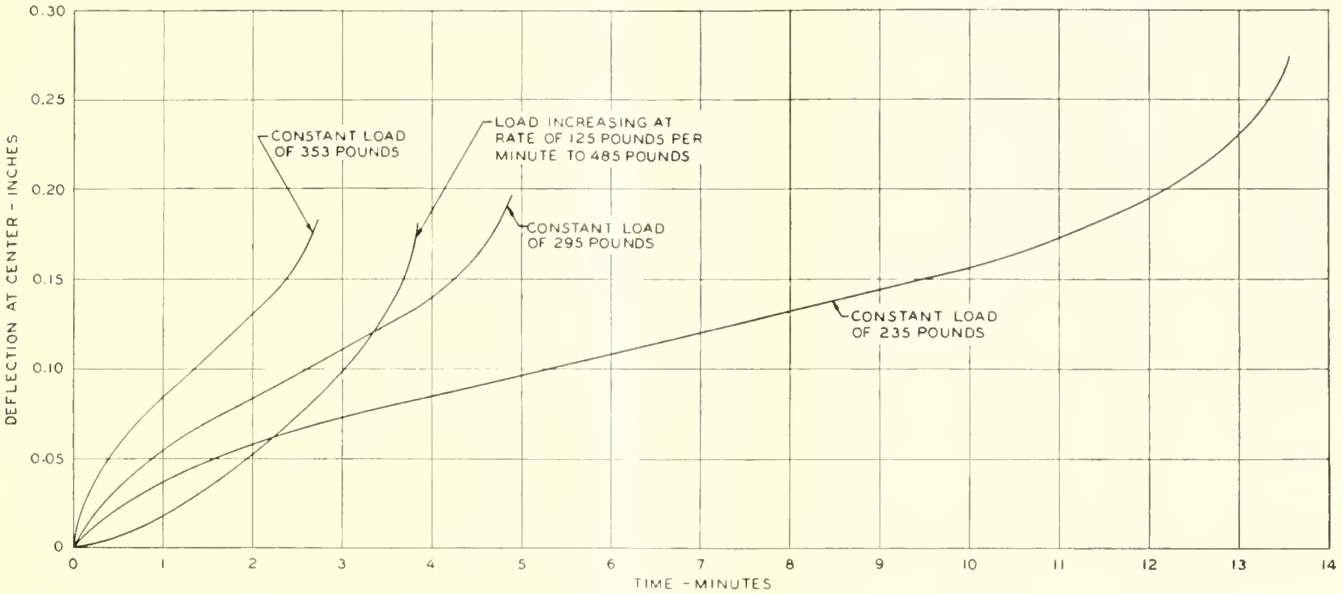


Figure 4.—Time-Deflection Curves for Beams Subjected to Constant and Increasing Loads.

The results of tests on beams with various depths and test spans are given in table 2. In order to show the amount of variation among the individual tests all the results are given. For the sizes of beams used in these tests the depth and test span do not appear to affect greatly the value of the modulus of rupture. The short, deep beams, however, have lower values for modulus of elasticity than have the longer and thinner beams. Part, if not all, of this variation in modulus of elasticity is probably caused by assuming that all the deflection is caused by bending stresses, whereas with deep beams of short spans a considerable amount of deflection is caused by shearing stresses.

Figure 5 shows that increasing the density of the specimens by greater compaction increases both their strength and stiffness.

Table 1.—*Effect of width of test specimen on the results of bending tests at 39° F.*

Width of specimen	Modulus of rupture	Modulus of elasticity
Inches	*Lb. per sq. in.*	*Lb. per sq. in.*
2.0	660	32,000
1.5	690	32,000
1.0	630	35,000

Figures 6 and 7 show the effect of temperature upon the test results. Temperatures as low as 0° F. were obtained using mixtures of ice, salt, and water. The temperature of −27° F., shown in figure 7, was obtained in a refrigerator. The specimens were kept in the refrigerator for 3 hours and then tested, the bath of the testing apparatus being filled with ice, salt, and water at 0° F. Because of the large amount of ice required to obtain temperatures below 39° F. it was difficult to place the specimens correctly in position for testing. The results, therefore, are not as consistent as they are at temperatures of 39° F. and higher. Nevertheless, they do show in a general way how a reduction in temperature increases the strength and stiffness of a mixture.

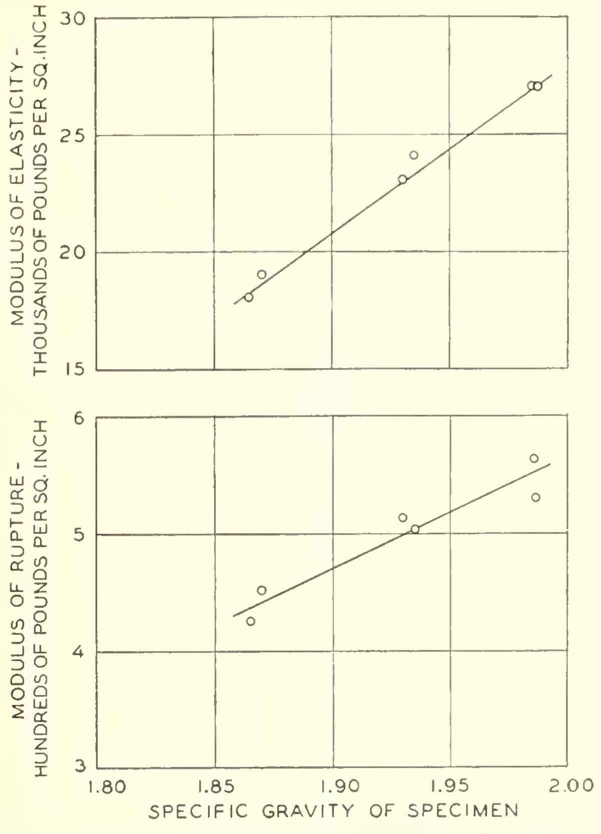


Figure 5.—Effect of Variations in Density Upon Modulus of Rupture and Modulus of Elasticity of Specimens.

EFFECT OF COMPOSITION ON BEAM STRENGTH INVESTIGATED

Mixtures were made with four different percentages of asphalt and four different ratios of sand and filler. All specimens were 2 inches deep molded under a pressure of 3,000 pounds per square inch. Specimens were tested at 39° F. with a span of 8 inches, applying the load at 120 pounds per minute, which gave a rate of stress of 180 pounds per square inch per minute. Results are given in table 3 and in figures 8, 9, and 10.

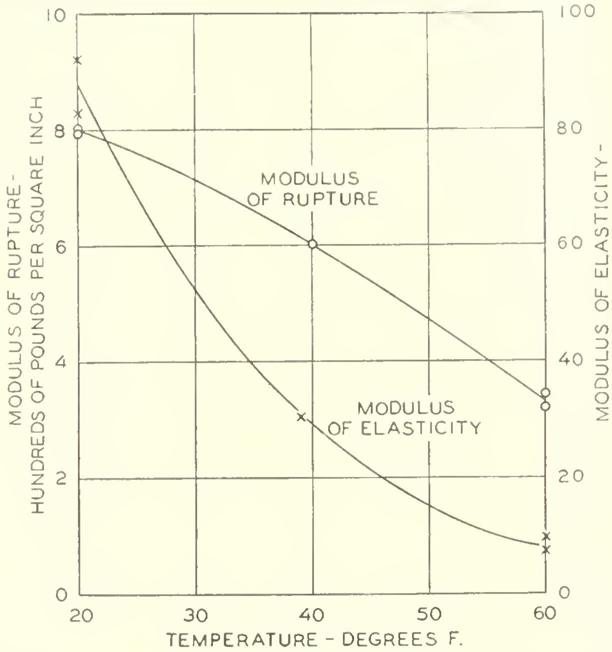


FIGURE 6.—EFFECT OF TEMPERATURE UPON MODULUS OF RUPTURE AND MODULUS OF ELASTICITY OF SPECIMENS.

TABLE 2.—*Effect of beam dimensions on the results of bending tests at 39 ° F. when the rate of stress*[1] *was 550 pounds per square inch per minute*

Span	Depth					
	1 inch		1½ inches		2 inches	
	Modulus of rupture	Modulus of elasticity	Modulus of rupture	Modulus of elasticity	Modulus of rupture	Modulus of elasticity
	Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
6 inches	688	51, 000	683	36, 000	774	34, 000
	611	39, 000	720	32, 000	771	34, 000
	712	45, 000	790	46, 000	756	----------
	768	47, 000	740	46, 000	844	48, 000
	702	----------	818	50, 000	816	33, 000
	896	60, 000	878	50, 000	864	42, 000
	762	52, 000	818	34, 000	764	33, 000
	820	56, 000	806	36, 000	792	33, 000
Average	745	50, 000	782	41, 000	797	37, 000
8 inches	765	56, 000	762	50, 000	781	50, 000
	803	66, 000	755	63, 000	823	57, 000
	795	73, 000	815	66, 000	832	53, 000
	765	63, 000	809	61, 000	754	45, 000
Average	782	64, 000	785	60, 000	798	52, 000
10 inches	714	50, 000	683	61, 000	739	54, 000
	721	50, 000	693	59, 000	828	62, 000
	787	82, 000	784	62, 000	756	54, 000
	758	70, 000	758	62, 000	758	54, 000
Average	745	63, 000	730	61, 000	770	56, 000

[1] The rate of stress is the modulus of rupture divided by the time required to break the beam.

For mixtures containing 0, 5, and 10 percent dust the modulus of rupture increased with increasing amounts of asphalt up to 14 percent, but for mixtures containing 15 percent dust the maximum modulus of rupture was obtained with 12 percent asphalt.

Figure 9 shows the composition of the specimens by volume and indicates why the modulus of rupture decreased when the mixture contained 14 percent asphalt with 15 percent filler in the aggregate. With 0, 5, and 10 percent filler in the aggregate the increase of asphalt from 8 to 14 percent increased the percentage of aggregate in the specimen. With 15 percent filler in the aggregate the percentage of aggregate by volume increased at a uniform rate with increases in asphalt up to 12 percent, but with 14 percent asphalt the percentage of the aggregate was decreased.

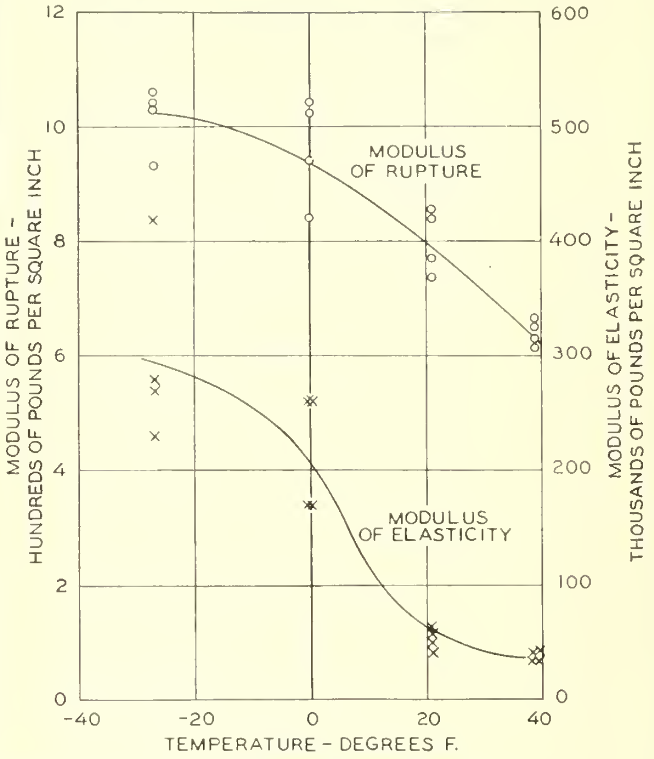


FIGURE 7.—EFFECT OF TEMPERATURE UPON MODULUS OF RUPTURE AND MODULUS OF ELASTICITY OF SPECIMENS.

TABLE 3.—*Results of bending tests on specimens*[1] *made with various amounts of asphalt and filler*

	Percentage of asphalt	Percentage of filler in aggregate			
		0	5	10	15
Modulus of rupture……lb. per sq. in.	8	385	450	490	570
Modulus of elasticity……do.		20, 000	26, 000	31, 000	38. 000
Total deflection……inches		0. 185	0. 170	0. 140	0. 130
Modulus of rupture……lb. per sq. in.	10	465	555	600	710
Modulus of elasticity……do.		22. 000	26, 000	31, 000	44, 000
Total deflection……inches		0. 190	0. 190	0. 165	0. 140
Modulus of rupture……lb. per sq. in.	12	555	650	780	915
Modulus of elasticity……do.		22, 000	28, 000	35, 000	43, 000
Total deflection……inches		0. 210	0 220	0. 190	0. 190
Modulus of rupture……lb. per sq. in.	14	785	865	895	850
Modulus of elasticity……do.		24, 000	25, 000	25, 000	22, 000
Total deflection……inches		0. 300	0. 330	0. 330	0 330

[1] 2-inch by 2-inch specimens tested with span of 8 inches. Test temperature, 39° F. Rate of stress, 180 pounds per square inch per minute.

Figure 10 shows the modulus of elasticity and the total deflection of the various mixtures. There is little consistent difference between the moduli of elasticity of the specimens containing, 8, 10, and 12 percent asphalt. All became stiffer as the amount of dust was increased. As the strengths of the specimens increased with increasing asphalt contents, the total deflections increased likewise although the moduli of elasticity were about the same. With 14 percent asphalt the modulus of elasticity was about the same for 0, 5, and 10 percent dust but decreased with 15 percent dust. With 10 and 15 percent dust the modulus of elasticity was much lower than that of specimens containing 8, 10, and 12 percent asphalt. The total deflection of the specimens containing 14 percent asphalt was much greater than the deflection of the specimens containing less asphalt.

TABLE 4.—*Results of bending tests on various types of surfaces [1] from Ohio projects*

Specimen No.	Type of surface	Age	Condition of surface	Rate of stress	Modulus of rupture	Modulus of elasticity
		Years		*Lb. per sq. in. per min.*	*Lb. per sq. in.*	*Lb. per sq. in.*
43309-1	Sheet asphalt on binder course.		Good	141	962	36,000
43309-2				143	1,040	37,000
43309-3				145	1,037	40,000
Average					1,015	38,000
43319-1	Two-course bituminous concrete.	1½	Excellent	145	880	84,000
43319-2				149	960	92,000
43319-3				145	840	92,000
Average					895	89,000
43321-1	do	4+	Good	138	818	61,000
43321-2				139	910	105,000
43321-3				139	854	101,000
Average					860	89,000
43322-1	do	2½	Excellent	228	850	83,000
43322-2				262	812	103,000
Average					830	93,000
43306-1	do	3+	Good	150	724	92,000
43306-2				140	718	103,000
43306-3				144	852	100,000
43306-4				135	902	96,000
Average					800	98,000
43320-1	do	4+	Generally cracked.	143	712	82,000
43320-2				145	738	137,000
43320-3				142	704	108,000
Average					720	109,000
43318-1	do	4+	do	136	648	56,000
43318-2				142	668	40,000
Average					660	48,000
43311-1	Sheet asphalt on binder course.	6½	do	140	524	24,000
43311-2				142	620	84,000
43311-3				142	617	96,000
43311-4				140	554	94,000
Average					580	75,000
43304-1	Two-course bituminous concrete.	3+	Badly cracked and worn.	140	489	112,000
43304-2				142	577	76,000
43304-3				140	632	102,000
43304-4				143	520	103,000
Average					555	98,000
43326-1	do	5½	Badly cracked.	140	520	103,000
43326-2				142	566	80,000
43326-3				141	576	105,000
43326-4				140	505	56,000
Average					540	86,000
43323T-1	Bituminous concrete.	2+	Good	140	536	78,000
43323T-2				143	454	90,000
Average					495	84,000
43323B-1	Bituminous concrete (base for above).			177	580	100,000
43323B-2				161	448	100,000
Average					515	100,000
43316-1	Two-course bituminous concrete.	3½	Badly cracked and ravelled.	141	462	102,000
43316-2				145	335	153,000
43316-3				141	522	161,000
43316-4				140	475	115,000
Average					450	133,000

[1] Tested at 39° F. Span, 6 inches.

In order to obtain information about the characteristics of actual pavements when tested in this manner, samples were obtained from several pavements in Ohio and Wisconsin and from one pavement in the District of Columbia.

The samples from Ohio were representative of several types of bituminous surfacing. The specimens were 2 inches wide and 2 inches deep. Each specimen contained all of the top course and a portion of the base course. They were tested at 39° F. on a span of 6 inches with the wearing surface in tension. The results are given in table 4. Samples are arranged in order of strength. It is, of course, difficult to correlate laboratory results with field behavior as there are so many unknown factors contributing to the failure of a surface. All of the pavement samples were taken from locations where the base was in good condition. Generally, the good pavements had the highest modulus of rupture but there seems to be little correlation between the modulus of elasticity and the condition of the pavement.

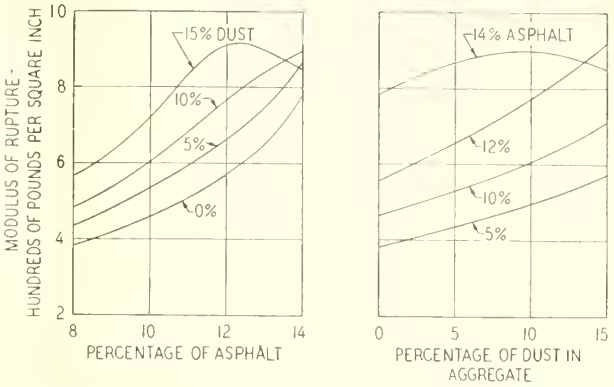


FIGURE 8.—EFFECT OF COMPOSITION UPON THE MODULUS OF RUPTURE.

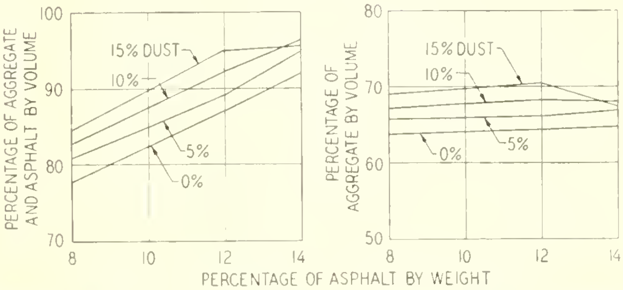


FIGURE 9. COMPARISON OF WEIGHT AND VOLUME COMPOSITION OF BEAMS.

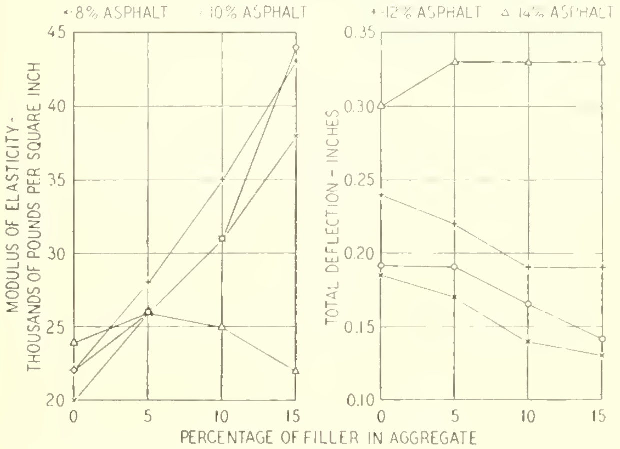


FIGURE 10.—EFFECT OF COMPOSITION UPON MODULUS OF ELASTICITY AND TOTAL DEFLECTION.

The samples from Wisconsin and the District of Columbia were of sheet asphalt and in most cases the binder course was sawed off before testing. These specimens were 2 inches wide and slightly over 1 inch thick. A few samples were tested with enough of the

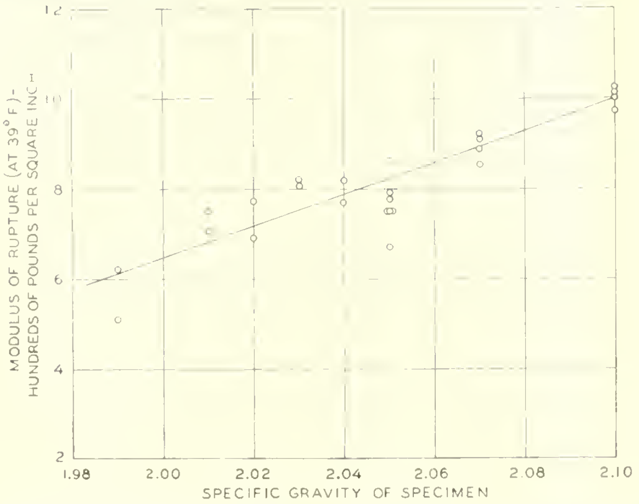


FIGURE 11.—EFFECT OF DENSITY UPON THE MODULUS OF RUPTURE OF SHEET ASPHALT FROM CONNECTICUT AVE., DISTRICT OF COLUMBIA.

binder course to make specimens 2 inches deep. All specimens were tested at 39° F. on a span of 6 inches with the wearing surface in tension. Results of these tests are given in tables 5, 6, and 7.

The samples from the District of Columbia were taken immediately after the pavement had been laid. They were all the same composition, varying only in density, and it is seen in figure 11 that there is a direct relationship between the density and strength of the specimens. Comparing tables 5 and 6 it is seen that for the 2-inch specimens the modulus of rupture is somewhat lower and the modulus of elasticity considerably lower than for the 1-inch specimens. A great part of this difference is caused by the difference in the rate of stress during the two tests. The pavement from which these samples were taken was in good condition in September 1939 after 3½ years service.

All of the Wisconsin samples were from pavements in good condition. Differences in composition account for the differences in strength and stiffness.

TABLE 5.—*Results of bending tests on samples* [1] *of sheet asphalt from Connecticut Ave., Washington, D. C.*

Section No.	Specific gravity	Modulus of rupture	Total deflection	Modulus of elasticity
		Lb. per sq. in	*Inches*	*Lb. per sq. in.*
A-1	1.99	619	0.090	65,000
		514	.060	68,000
A-2	2.05	777	.095	67,000
		671	.070	95,000
C-1	2.02	774	.100	49,000
		688	.090	61,000
	2.04	813	.090	76,000
		770	.095	65,000
C-2	2.05	753	.095	68,000
		791	.090	68,000
		752	.075	80,000
		862	.080	83,000
D-1	2.07	887	.120	62,000
		917	.105	70,000
		912	.115	77,000
		856	.110	64,000
D-2	2.10	968	.100	72,000
		1,015	.090	84,000
		1,010	.100	82,000
		1,016	.095	82,000
D-3	2.01	749	.100	63,000
		706	.080	66,000
	2.03	805	.110	62,000
		820	.090	83,000

[1] Top course only, 1.1 to 1.2 inches thick, 2 inches wide. Tested at 39° F. Span, 6 inches. Rate of stress, 425 pounds per square inch per minute.

TABLE 6.—*Results of bending tests on samples* [1] *of sheet asphalt from Connecticut Ave., Washington, D. C.*

Section No.	Specific gravity	Modulus of rupture	Total deflection	Modulus of elasticity
		Lb. per sq. in.	*Inches*	*Lb. per sq. in.*
C-1	2.16	778	0.080	48,000
	2.16	758	.090	41,000
C-2	2.17	747	.085	39,000
	2.17	792	.080	44,000
D-1	2.14	721	.095	34,000
D-2	2.15	882	.100	35,000
D-3	2.12	618	.080	37,000

[1] Top and binder course sawed to thickness of 2 inches. Tested at 39° F. Span, 6 inches. Rate of stress, 150 pounds per square inch per minute.

TABLE 7.—*Results of bending tests on samples* [1] *of sheet asphalt from Wisconsin*

Specimen No.	Rate of stress	Modulus of rupture	Total deflection	Modulus of elasticity
	Lb. per sq. in. per min.	*Lb. per sq. in.*	*Inches*	*Lb. per sq. in.*
43331	450	1,125	0.060	128,000
	465	1,220	.075	96,000
43332	460	1,069	------	120,000
	455	1,308	.075	110,000
43335	420	1,039	.055	119,000
	420	1,034	.055	117,000
43341	435	1,142	------	138,000
43342	490	1,244	.110	98,000
	490	1,233	.120	98,000
43343	480	1,039	.090	103,000
	480	1,120	.090	109,000
43333	425	1,094	.105	70,000
	425	1,117	.080	80,000
3334	445	1,142	.080	101,000
43344	470	1,174	.110	77,000
43345	470	1,116	.125	74,000
43346	470	1,105	.075	106,000
	485	1,183	.090	106,000
43347	465	1,130	.105	88,000
	430	1,106	.095	78,000
43336	465	1,020	.150	66,000
	445	983	.140	63,000
43337	430	891	.115	65,000
43338	430	888	.100	77,000
	465	1,004	.115	86,000
43339	430	944	.160	58,000
	420	939	.170	55,000
43340	465	990	.120	87,000

[1] Top course only, 1.1 to 1.2 inches thick. Tested at 39° F. Span, 6 inches.

These results of tests on actual surfaces are reported only as an indication of the possibilities of this type of investigation.

SUMMARY

These flexure tests of bituminous mixtures with a constant rate of loading show that the modulus of rupture and the modulus of elasticity both increase as the rate of stress increases.

With a constant rate of stress it is indicated:

1. That the value of the modulus of rupture is not greatly influenced by the ratio of span length to depth of the test specimen.
2. That for a given span length the value of the modulus of elasticity decreases as the depth of beam increases and that for a given depth of beam the modulus increases as the span length is increased. In other words the modulus of elasticity decreases as the ratio of depth of beam to span length increases.
3. That the modulus of rupture and the modulus of elasticity, calculated by the usual formulas, are satisfactory measures of the strength and stiffness of bituminous mixtures when due consideration is given to the effect on the modulus of elasticity of the ratio of span length to depth of beam.
4. That increasing the density of specimens by increased compaction increases their strength and stiffness.

(*Continued on p. 81*)

EFFECT OF CONSISTENCY AND TYPE OF ASPHALT ON THE HUBBARD-FIELD STABILITY OF SHEET ASPHALT MIXTURES

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by W. O'B. HILLMAN, Assistant Highway Engineer

BECAUSE the consistency of the contained asphalt affects the resistance of bituminous mixtures to displacement under traffic, it has been customary to vary somewhat the grade of asphalt depending on the climatic and traffic conditions. In general, the grades used in the Northern States are softer than those used in the Southern States for the same type of construction. In the same locality harder grades are used on the streets carrying the heavier traffic.

Recently, because of the great amount of cracking occurring in pavements, there has been a general trend towards the use of softer asphalts in sheet asphalt and asphaltic concrete construction. The effect of these softer grades of asphalt on the stability is, therefore, a matter of present interest. The work here described gives the results of Hubbard-Field stability tests [1] on a typical sheet asphalt aggregate mixed with various grades and types of asphalt.

The mixture was similar in composition to that currently used in the District of Columbia. It contained 78 percent Potomac River sand, 11 percent limestone dust, and 11 percent asphalt. The first series of tests were made with 10 different grades of Mexican asphalt ranging in penetration from 23 to 182. These asphalts were all made by the same producer using the same process of manufacture.

The penetrations of these asphalts were determined at a number of temperatures. The relations between penetration values and test temperatures are shown in figure 1.

As has already been shown [2] the slope of the log penetration-temperature curve is a measure of the temperature susceptibility of the asphalt. The greater the slope the more susceptible is the asphalt. The slope can be calculated as follows:

$$\text{Slope} = M = \frac{\log p_2 - \log p_1}{t_2 - t_1}$$

where p_2 and p_1 are penetrations at temperatures t_2 and t_1, respectively. For these Mexican asphalts the slope had a constant value of approximately 0.022.

By extending the curves upward, as represented by the dotted lines in figure 1, the penetration of the asphalts at any temperature may be estimated. Plotted on this dotted portion are the softening points of the asphalts. The penetration at the softening point averaged about 850 for these asphalts.

In order to minimize alteration of the asphalt during preparation of the specimens, the following method of fabrication, which previous work had shown to cause very little hardening of the asphalt, was followed. Sufficient sand and filler to make three specimens were combined at room temperature. The cold asphalt was added and the pan containing the combined materials placed on a hot plate. Mixing was accomplished by continuous stirring at the lowest possible temperature. The temperature of the finished mixtures ranged from about 220° F. for the 180-penetration asphalt to about 280° F. for the 23-penetration asphalt.

STABILITIES OF MEXICAN ASPHALTS DEPENDED UPON THEIR CONSISTENCIES

Two forming molds and two compression machines were used, and after mixing was completed two specimens were immediately compressed using a load of 3,000 pounds per square inch maintained for 2 minutes. As soon as the first two specimens had been compacted the mixture for the third specimen, which had been kept hot, was placed in the mold and compacted. Thus only about 4 minutes elapsed between the completion of mixing and the completion of molding. There was found to be no difference in the density or the stability of the three specimens comprising a set.

The specimens were tested for Hubbard-Field stability at 122° and 140° F. The results are given in table 1, together with the softening point and penetration of the asphalt and the estimated penetration of the asphalt at 140° and 122° F. When the logarithm of the stability is plotted against the logarithm of the penetration at 77° F. of the contained asphalt, as in figure 2, all points fall approximately on two parallel straight lines, one for a test temperature of 140° F. and the other for a test temperature of 122° F.

For these Mexican asphalts doubling the penetration of the asphalt reduced the Hubbard-Field stability of the sheet asphalt mixture by about 25 percent.

When the logarithm of the stability at either 122° or 140° F. is plotted against the logarithm of the penetration of the asphalt at the same test temperature, as in figure 3, all points fall approximately on the same straight line. Thus for asphalts from the same source and of the same type, the stability is dependent only upon the consistency of the asphalt at the test temperature.

With materials such as these, all of which are equally susceptible to temperature and have approximately the same consistency at their softening points, the softening point is a measure of consistency.

When the logarithm of the stability is plotted against the softening point of the contained asphalt (fig. 4), all points fall approximately on two parallel straight lines, one for stability at 140° F. and the other for stability at 122° F.

All of these asphalts had approximately the same consistency at their softening points. Because they were

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[1] A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures, by Prevost Hubbard and F. C. Field. Proceedings, American Society for Testing Materials, vol. 25, pt. II, p. 335 (1925).

A Study of Certain Factors Affecting the Stability of Asphalt Paving Mixtures, by Prevost Hubbard and F. C. Field. Proceedings, American Society for Testing Materials, vol. 26, pt. II, p. 577 (1926).

[2] The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades. R. H. Lewis and J. Y. Welborn. PUBLIC ROADS, vol. 21, No. 1, March 1940.

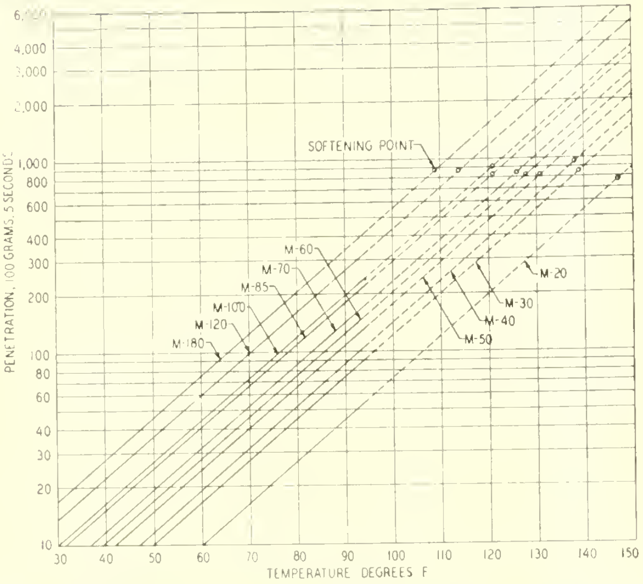

FIGURE 1.—RELATION BETWEEN TEMPERATURE AND PENETRATION FOR THE MEXICAN ASPHALTS USED.

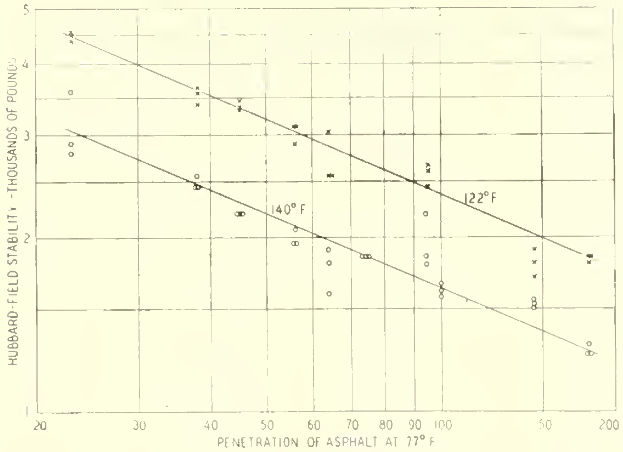

FIGURE 2.—RELATION BETWEEN HUBBARD-FIELD STABILITY AT 122° F. AND 140° F. AND THE PENETRATION AT 77° F., OF THE CONTAINED MEXICAN ASPHALT.

equally susceptible to changes in temperature they would have approximately the same penetration at temperatures that are above or below their respective softening points by equal amounts. When the penetration test temperature is subtracted from the softening point, asphalts having the same difference have the same consistency, and mixtures made from them have approximately the same stability, as shown in figure 5. This figure also shows that for these Mexican asphalts and these proportions, increasing the temperature of the stability test by 10° F. decreases the stability by about 20 percent.

In order to determine whether or not asphalts from other sources behaved in the same manner, a second series of specimens was made using two grades of petroleum asphalt from each of five different producers. The characteristics of these asphalts are given in table 2. The specimens were tested for stability at 104°, 122°, and 140° F., the results being given in table 3.

In figure 6 the stabilities of these mixtures at all three temperatures are plotted against the penetrations of the asphalts at 77° F. Asphalts from each producer are plotted separately.

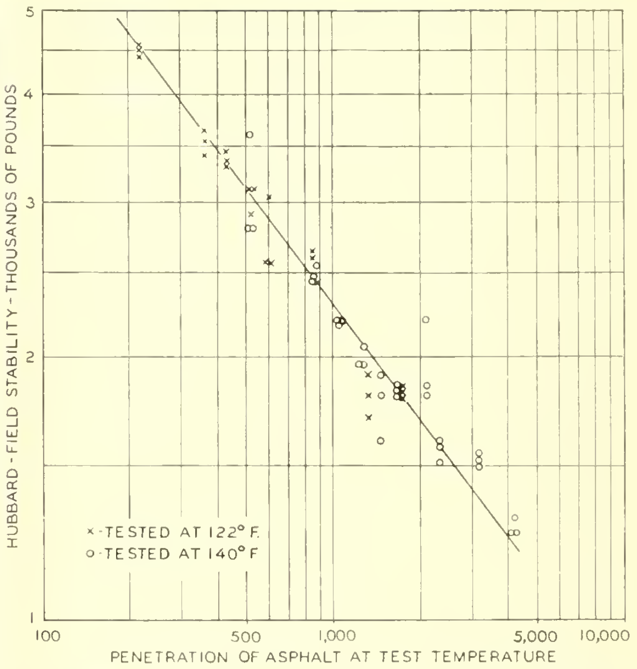

FIGURE 3.—RELATION BETWEEN STABILITY AND THE PENETRATION OF THE ASPHALT AT TWO TEST TEMPERATURES.

TABLE 1.—*Hubbard-Field stability of mixtures containing various grades of Mexican asphalt*

Identification	Softening point, ring and ball	Penetration at 77° F.	Estimated penetration at—		Hubbard-Field stability at—	
			140° F.	122° F.	140° F.	122° F.
	°F.				*Pounds*	*Pounds*
M-20	147	23	525	215	2,800 2,900 3,600	4,500 4,550 4,400
M-30	139	38	880	360	2,500 2,450 2,450	3,550 3,650 3,400
M-40	138	45	1,050	430	2,200 2,200 2,200	3,450 3,350 3,300
M-50	131	56	1,260	515	2,050 1,950 1,950	2,900 3,100 3,100
M-60	128	64	1,470	600	1,600 1,900 1,800	3,050 2,550 2,550
M-70	126	74	1,680		1,850 1,850 1,850	
M-85	121	95	2,100	860	2,200 1,800 1,850	2,450 2,650 2,600
M-100	121	100	2,300		1,600 1,580 1,650	
M-120	114	144	3,150	1,290	1,500 1,550 1,530	1,800 1,700 1,900
M-180	109	182	4,200	1,720	1,250 1,300 1,250	1,800 1,850 1,850

SUSCEPTIBILITY TO TEMPERATURE AND BONDING STRENGTH OF ASPHALT DETERMINE STABILITY OF MIXTURE

Figure 7 shows the relation between the stability at 140° F. and the penetration of the asphalt at 77° F. for all mixtures including those made with the Mexican asphalts. In every case the softer grades of asphalt gave lower stability values than the harder grades of asphalt from the same producer, but the decrease in stability was not the same for all the samples. Stability values from figure 7 show that the stability of specimens made with 100-penetration asphalts varied from 69 to 91 percent of the stability of specimens made with 50-penetration asphalt from the same producer.

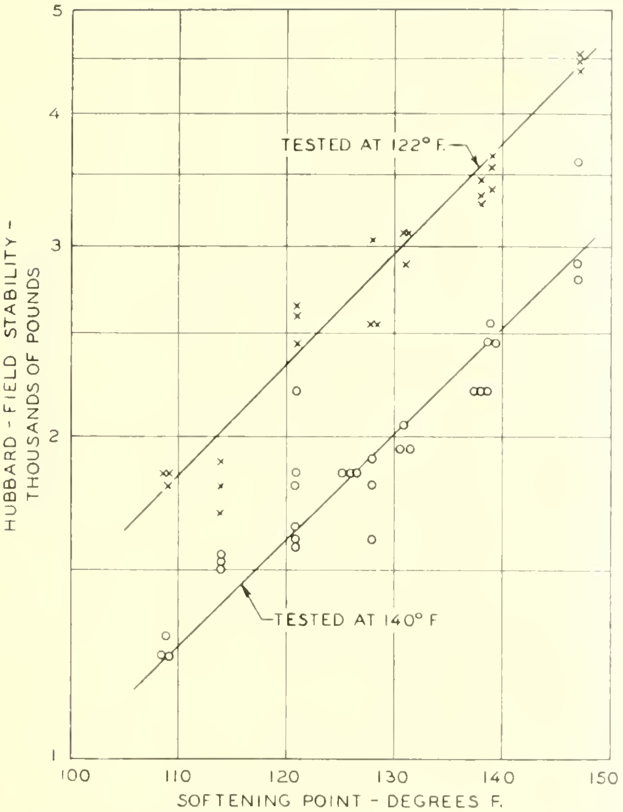

FIGURE 4.—RELATION BETWEEN STABILITY AT 122° F., AND 140° F., AND THE SOFTENING POINT OF THE CONTAINED MEXICAN ASPHALT.

TABLE 2.—*Characteristics of the asphalts used in the second series of tests*

Identification	Penetration at 77° F.	Softening point	Slope of penetration curve	Penetration at softening point
		°F.		
A-50	61	118	0.031	1,060
A-85	96	111	.031	1,120
B-50	58	137	.017	630
B-85	90	121	.019	620
C-50	57	130	.022	750
C-85	97	115	.023	720
D-50	60	120	.029	1,080
D-85	91	112	.028	840
E-50	50	123	.024	630
E-85	87	113	.026	760

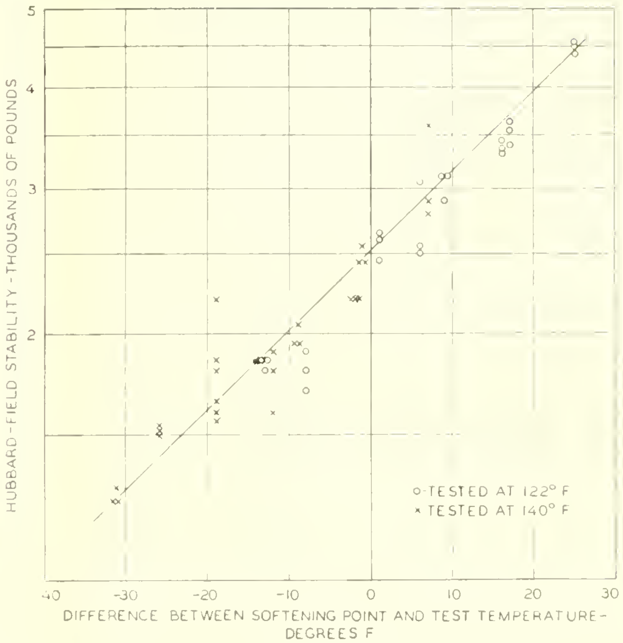

FIGURE 5.—RELATION BETWEEN THE HUBBARD-FIELD STABILITY AND THE TEMPERATURE DIFFERENCE BETWEEN THE SOFTENING POINT OF THE CONTAINED BITUMEN AND THE TEST TEMPERATURE.

TABLE 3.—*Hubbard-Field stability of mixtures containing two grades of asphalt from various producers*

Identification	Penetration at 77° F.	Estimated penetration at—			Hubbard-Field stability at—		
		140° F.	122° F.	104° F.	140° F.	122° F.	104° F.
					Pounds	*Pounds*	*Pounds*
A-50	61	5,000	1,400	390	1,050 1,250 1,250	1,950 1,770 2,020	3,150 3,200 3,300
A-85	96	9,000	2,450	660	1,100 1,050 1,100	1,750 1,580 1,500	2,700 2,650 2,800
B-50	58	720	350	170	1,700 1,700 1,700	2,850 2,850 2,600	4,600 5,000 4,300
B-85	90	1,400	650	295	1,400 1,400 1,300	2,500 1,950 2,050	3,300 3,100 3,500
C-50	57	1,450	590	230	1,600 1,600 1,550	2,500 2,520 2,250	4,000 3,750 3,900
C-85	97	2,650	1,050	400	1,100 1,250 1,250	1,900 1,860 1,900	3,250 3,050 3,200
D-50	60	4,200	1,300	380	1,250 1,150 1,200	2,020 2,150 1,750	3,500 3,250 3,050
D-85	91	5,000	1,600	500	1,050 1,200 1,150	1,750 1,650 1,550	2,650 2,650 2,750
E-50	50	1,600	600	225	1,450 1,900 1,300	2,470 2,000 2,350	4,550 4,000 4,500
E-85	87	4,000	1,350	440	1,100 1,100 1,100	1,840 2,100 1,800	3,200 3,150 3,150

TABLE 4.—*Stabilities of asphaltic mixtures tested at temperatures at which the asphalt would have a penetration of 1,000*

Identification	Stability	Identification	Stability
	Pounds		*Pounds*
M	2,300	C	1,900
A	2,250	D	2,100
B	1,500	E	2,000

They also show that the stability of specimens made with 50-penetration asphalt D was only 56 percent of the stability of specimens made with 50-penetration asphalt M. Thus there was a greater percentage difference in the stability of specimens made with 50-penetration asphalt from different producers than in the stability of specimens made with 50- and 100-penetration asphalts from the same producers.

In figure 8 the logarithm of the stability of the second series of mixtures is plotted against the logarithm of the penetration of the 50- and 85-penetration asphalts at the temperature of the stability test. In this figure, as in figure 3 for the Mexican asphalts, the points for asphalts from the same producer fall approximately upon the same straight line.

Table 4 gives the stabilities of mixtures containing each asphalt at the temperature at which the asphalt had an estimated penetration of 1,000. Values for this table are taken from figures 3 and 8. Since all the asphalts had the same consistency, the differences in stability values must have been caused by some in-

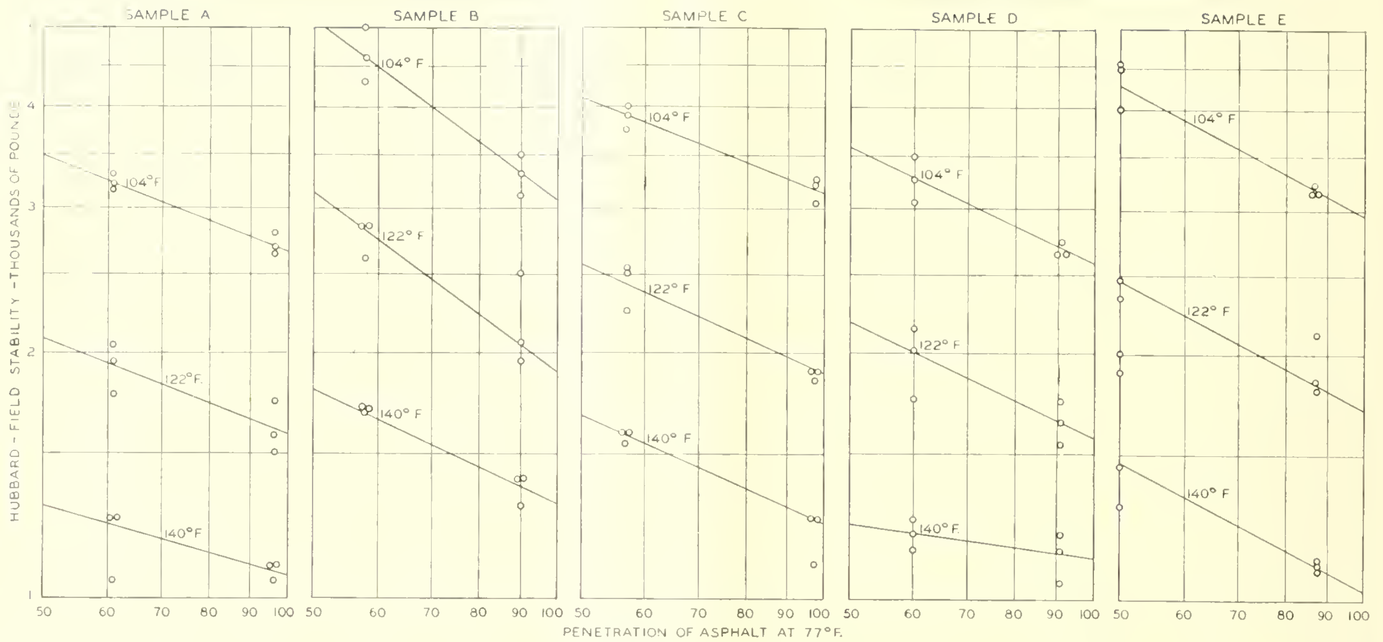

FIGURE 6.—RELATION BETWEEN STABILITY AT VARIOUS TEMPERATURES AND THE PENETRATION AT 77° F. OF THE VARIOUS TYPES OF CONTAINED ASPHALT.

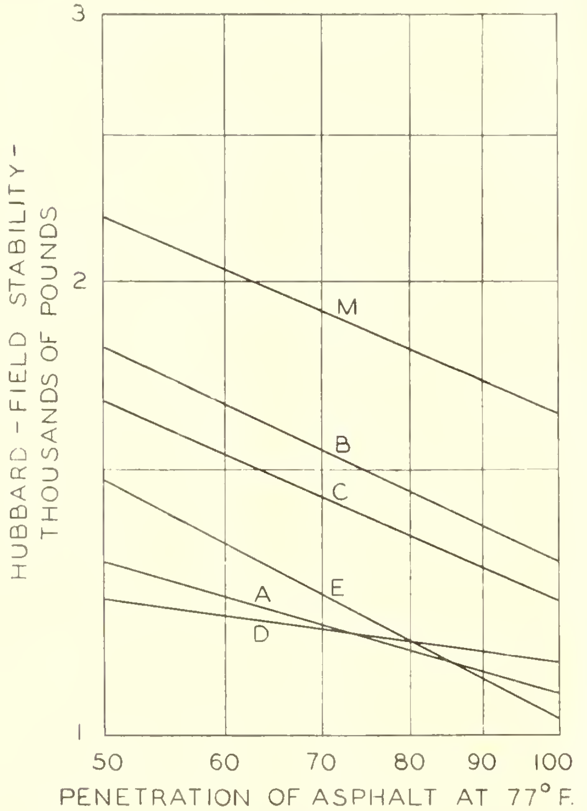

FIGURE 7.—RELATION BETWEEN STABILITY AT 140° F. AND THE PENETRATION AT 77° F. OF THE CONTAINED ASPHALT.

herent quality of the asphalt which, in this report, is called bonding strength. Thus asphalt B may be assumed to have the least bonding strength, while asphalts M and A had the greatest bonding strength.

There are then two characteristics of an asphalt which determine the stability at any temperature of mixture containing it; its susceptibility to temperature, and its bonding strength. For example, table 4 shows asphalts A and M to have about the same bonding

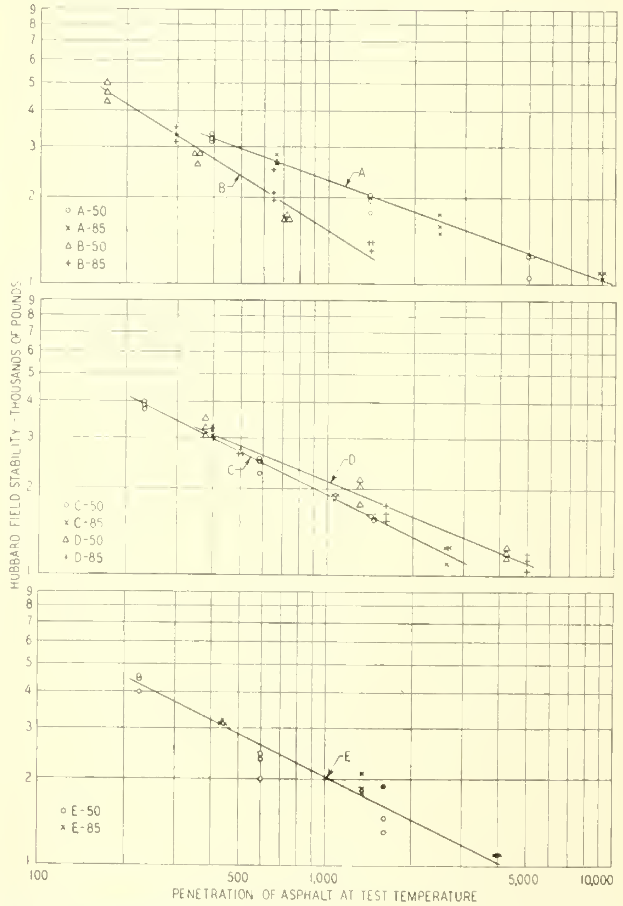

FIGURE 8.—RELATION BETWEEN STABILITY AND THE PENETRATION AT TEST TEMPERATURE OF THE VARIOUS CONTAINED ASPHALTS.

(Continued on page 82)

THE DETERMINATION OF SODIUM AND POTASSIUM IN CEMENTS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by L. G. CARMICK, Associate Chemist

IN ENDEAVORS to find reasons for the often erratic behavior of portland cements, interest has turned recently toward minor constituents that are commonly ignored in analysis. There is particular interest in knowing to what extent sodium and potassium compounds are present. These elements and their compounds, which for convenience may be referred to simply as alkalies, seem to be present in all cements to the extent of at least 0.3 or 0.4 percent, and cements made in some parts of the United States contain appreciably over 1 percent. What effect alkalies have on the behavior of cements is still uncertain, but there is reason for believing that they do exert an influence. It therefore seems likely that determination of alkalies will be a more frequent task in the future than it has been heretofore.

The method that has always been relied on for the determination of alkalies is that proposed many years ago by J. Lawrence Smith. This method was intended primarily for use in the analysis of igneous rocks, which are very difficult to bring into solution. While it is capable of yielding excellent results in the hands of a careful analyst, the method has several disadvantages that may easily lead to error. In the first place, it is necessary to use large amounts of reagents, particularly calcium carbonate which is often impure. This makes it necessary to run blanks and make corrections. Secondly, in the long heating or "fusion" there is some uncertainty that all the alkali has been rendered soluble, and if the temperature has been too high some of the alkali may have been volatilized. Thirdly, there are large amounts of insoluble matter to wash; and finally, the method is tedious and requires constant attention.

NEW METHOD GIVES ACCURATE RESULTS

For these reasons a better method was sought. The following procedure was found to be quite satisfactory and largely free from the objections just noted. It has now been tried out by several analysts and found to give very accurate results.

While any reasonable quantity of cement may be used, it is convenient to take 15.625 grams. Put this in a beaker or evaporating dish and add enough water to make a thin grout, then add enough hydrochloric acid to decompose the cement completely. Evaporate to dryness on the hot plate but do not bake. Add about 300 milliliters of water and digest on the hot plate for half an hour, then heat to boiling. Most of the silica and R_2O_3 group will now be precipitated and in suspension. Add 2 or 3 drops of phenolphthalein indicator and then, a little at a time, some pure quicklime until the solution is strongly alkaline as shown by the pink color. Quicklime is an excellent precipitant for magnesia as well as for silica and the R_2O_3 group, and also the precipitate caused by lime is less voluminous than that caused by ammonia. The freshly ignited lime obtained in the ordinary analysis of cement does very well and about 1 gram is usually enough.

Boil gently for a few minutes, allow to cool somewhat, and transfer to a 500-milliliter volumetric flask. Add water to bring the volume to 500 milliliters, mix well, and allow the precipitate to settle. Decant through a filter just 400 milliliters of the supernatant liquid. This 400 milliliters should contain only lime and the alkalies from 12.5 grams of cement. After obtaining the 400 milliliters of filtrate, the residue and precipitate in the flask are discarded without further treatment.

To the filtrate add a solution of ammonium carbonate and ammonium hydroxide in sufficient amount to precipitate all the lime, avoiding unnecessary excess. Boil gently, allow to cool somewhat, transfer to a 500 milliliter volumetric flask, add water to bring the volume to 500 milliliter, mix well, and allow the lime to settle. Decant through a filter just 400 milliliter of the supernatant liquid, discard the residue, and precipitate in the flask. The filtrate thus obtained should contain only the alkalies from just 10 grams of cement together with ammonium salts and perhaps a very little lime. Add to this filtrate about 1 milliliter of concentrated sulphuric acid, evaporate in a platinum dish, and volatilize the ammonium salts. Dissolve the residue in 30 or 40 milliliters of water and add a little quicklime in order to throw down any magnesia that may possibly have escaped the first precipitation. Filter into a small beaker, add ammonia water and ammonium oxalate, warm to precipitate the last of the lime, and filter into a platinum dish. Add a drop or two of sulphuric acid, evaporate to dryness, and volatilize the ammonium salts. Heat to dull redness for a moment, cool, and weigh the sodium and potassium sulphates.

Recently the following modification has been tried and found to add to the convenience of the method in that it does away with the necessity of drying and volatilizing large amounts of ammonium salts.

Instead of precipitating the lime with ammonium carbonate, place the 400 milliliters of filtrate from the silica and R_2O_3 group in a 500-milliliter volumetric flask and add slightly more oxalic acid in hot concentrated solution than can combine with the lime that is present. The amount of oxalic acid is determined by multiplying the number of grams of CaO by 2.25 and adding 1 gram more. Add enough water to make up 500 milliliters of solution, mix well, and allow the precipitate to settle. Although hydrochloric acid is present, nearly all of the lime will be precipitated. Decant through a filter 400 milliliters of the liquid, discard the residue, and precipitate in the flask. Evaporate the filtrate in platinum on the steam bath to practical dryness. Add at least 50 milliliters of water, digest well on the steam bath, make ammoniacal, and filter into a small beaker. Test the filtrate with ammonium oxalate to insure the absence of lime. Add a few drops of concentrated sulphuric acid, evaporate to dryness on the steam bath in platinum, and volatilize the ammonium salts, which in this case will be only a small amount. The residue in the dish should be the pure alkali sulphates; this may be verified by testing for the presence of traces of magnesium and calcium compounds.

From this point, the sodium and potassium may be separated in the usual manner by platinic chloride if it is desired to do so. If it be assumed that sodium and potassium are present in equal quantities, the amount of Na_2O plus K_2O is found by multiplying the weight of the mixed sulphates by 0.4884.

The amount of cement used for analysis is of course optional with the analyst. If 7.8125 grams are taken instead of double that amount as suggested herein the result will be the alkalies from 5 grams instead of 10. In such case it is quite feasible to use 250-milliliter volumetric flasks instead of the 500 size, and there is less liquid to evaporate.

The large precipitates that are discarded may seem at first thought to be a source of error, but it can be shown that their actual volume is such that the error caused thereby is negligible. It will be observed that a minimum of reagents is used in this method and those that are used are readily obtainable in very pure form. It will of course be understood that whenever water is referred to, distilled water of the highest purity is meant.

(Continued from page 68)

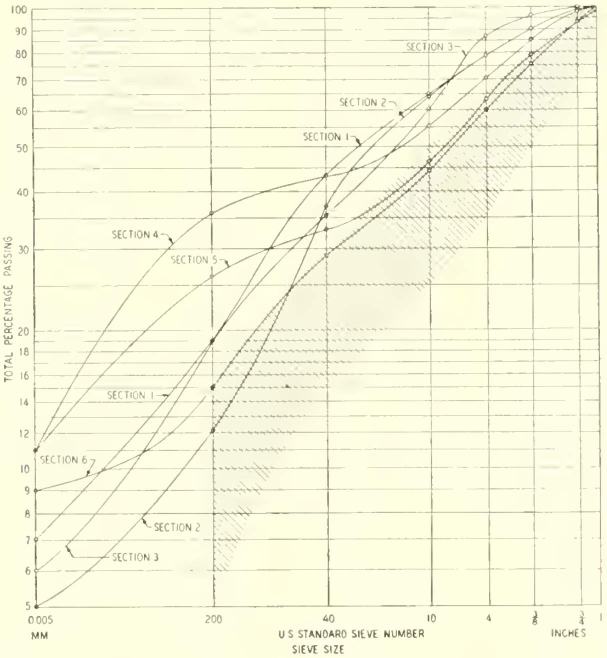

FIGURE 5.—GRADINGS OF MATERIALS. SHADED AREA INDICATES ZONE WITHIN WHICH ARE INCLUDED THE SPECIFICATION REQUIREMENTS OF THE A. A. S. H. O. FOR COARSE-GRADED TYPE AGGREGATE BASE-COURSE MATERIALS HAVING 1-INCH MAXIMUM SIZE.

however, contained hydrated lime, which has the property of increasing the liquid limits and reducing the plasticity indexes when added to many soils.

Previous investigations have shown that, with the ground-water elevation ½ inch above the top of the sub-base, concentrated traffic provides a condition that is sufficiently severe to enable identification of the definitely unsatisfactory materials. Using this as a criterion, the mixtures tested would be rated as shown in table 6.

TABLE 6.—*Rating of sections after testing under concentrated traffic, with water elevation ½ inch above the top of the sub-base*

Section No.	Composition	Admixture of hydrated lime	Service rating
		Percent	
1	Chert A-sand	0	Unsatisfactory.
2	Chert A-sand	5	Good.
3	Chert A-slag	0	Borderline.
4	Chert B	0	Complete failure.
5	Chert B	5	Unsatisfactory.
6	Chert C	5	Good.

The addition of hydrated lime to the sand-chert A mixture and to chert B resulted in improved performance of the materials when used as base courses. The results obtained in the track with a combination of basic granulated slag and chert A were superior to those with a combination of sand and chert A.

Section 1 became unstable with the water ½ inch above the top of the sub-base and had failed completely at 124,400 wheel-trips. Section 2, which differed from section 1 only by the addition of 5 percent of hydrated lime, gave excellent service as a base course throughout all phases of the testing. Section 3 withstood the 40,000 wheel-trips of concentrated traffic with water at the ½-inch level without excessive movement in the base. However, with continued traffic and increased elevation of the ground water, the vertical displacements continued to increase at a fairly uniform rate (see fig. 1) until at the end of the test the section was in an unsatisfactory condition, as shown in figure 4.

The grading and plasticity index of the mixture used in section 1 are indicated to be responsible for the failure of this section when tested as a base course. Section 2 was composed of the same materials with an admixture of hydrated lime. Its grading was only slightly better than that of section 1 and could not be responsible for the marked difference in behavior, but it was nonplastic with a plasticity index of 0. Similarly, the grading of the section 3 material was not widely different from that of section 1 but it had a plasticity index intermediate between those of sections 1 and 2. Its behavior under traffic was much superior to that of section 1, although not as good as that of section 2.

The tests (see table 5) made at the time of failure or the end of testing show that section 2 had 64.7 percent of solids by volume, or considerably less than either of the other two sections in which chert A was used. Therefore, the differences in behavior of the sections cannot be ascribed to differences in compaction, since previous investigations have led to the conclusion that, for a given material, increase in density would produce increase in stability.

Section 4, chert B, failed immediately when subjected to concentrated traffic with water in the sub-base. Section 5 (chert B with an admixture of hydrated lime) became slightly unstable under these testing conditions but did not fail completely until water was raised to the 4½-inch level and wheel loads were increased from 800 to 1,000 pounds. The gradings of the section 4 and section 5 materials were similar and the plasticity indexes were 2 and 0, respectively. Again the track density in the lime-treated section was less than that of the untreated section.

Section 6 consisted of a mixture of chert gravels with a 5 percent hydrated lime admixture. The grading of this mixture conformed with the A. A. S. H. O. specifications for base-course materials, yet each of the cherts came from a location on a highway where base failure had occurred. However, with the lime admixture this composite material gave good service in the track, except that near the end of the test a small, soft spot developed (see fig. 4). This resulted in the abrupt change in slope of the displacement curves shown in figure 1. It should be emphasized that this softening was confined to one spot and did not represent the entire section. The condition developed only under the most extreme testing conditions and is thought to have been caused by some construction feature, since samples from the rutted and intact portions of the section had almost identical compositions and densities.

CONCLUSIONS

The following conclusions appear to be justified:

1. Although only three chert gravels were investigated, there are strong indications that the behavior as base courses of this class of local materials can be greatly improved by the addition of a basic material such as hydrated lime or granulated slag. Other possible admixtures include limestone or slag screenings, lime wastes, and similar materials of a basic character which may be economically available.
2. It is indicated that the beneficial action of the basic admixture will be greater if the material is in a finely divided state, except where it is intended to perform the dual purpose of improving the grading and altering the physical characteristics of the material.
3. The addition of 5 percent of hydrated lime to a composite chert (sec. 6), conforming to the grading requirements of the A. A. S. H. O. specifications for base-course materials produced a mixture which gave good service when tested in the circular track even though all of the cherts which were combined had come from locations where base-course failures had been observed.
4. The addition of 5 percent of hydrated lime to the chert B which did not meet these grading requirements resulted in a marked improvement in the behavior of this material under test traffic. (Compare secs. 4 and 5.)
5. Lime also greatly improved the performance of the chert A to which sand had been added to reduce the plasticity index to approximately the upper limit of the A. A. S. H. O. specification. (Compare sections 1 and 2.)
6. Basic granulated slag added to chert A in an amount sufficient to lower the plasticity index to 5 resulted in a mixture that was more stable when tested in the circular track than the mixture of sand and chert A without hydrated lime. (Compare sections 1 and 3.)
7. Density as such is not a true measure of the stability of granular mixtures. For any one given material an increase in density is accompanied by an increase in stability, but for materials having different compositions, the one having the greatest density may or may not be the most stable. Thus in the present investigation, in all cases where a direct comparison is possible, the mixtures containing lime had lower densities than the corresponding sections without lime; yet their behavior in the track showed the sections with lime to be the more stable.
8. No specifications for chemically treated chert-gravel base-course materials can be prepared from the information so far developed. It is suggested, however, that this class of material be used only if the grading conforms to or closely approaches the A. A. S. H. O. specification for base-course materials, and if the admixture reduces the plasticity index of the fraction passing the No. 40 sieve to 3 or less. It is believed that if these conditions are met the liquid limit of acceptable mixtures containing basic ingredients may be as high as 35.

(Continued from page 74)

5. That decreasing the temperature of a mixture increases its strength and stiffness. It is probable, however, that there is some limiting temperature below which the strength and stiffness of a mixture would not be increased.
6. That increasing the amount of asphalt in a mixture up to the amount required to fill the voids increases its strength but has little effect upon the stiffness of the mixture. Increasing the amount of dust in a mixture up to the amount required to fill the voids increases both the strength and stiffness of the mixture.

As a result of these tests the following suggestions are made regarding a suitable test procedure:

1. A test temperature of 39° F. appears to have definite advantages. This temperature may be obtained and held constant with a bath containing very little ice. Temperatures below this are hard to maintain and require a large amount of ice which interferes with the correct placing of the specimen. At higher temperatures the strength and stiffness decrease and it would be difficult to distinguish between mixtures.
2. Preferably, the span length should be 10 inches for all depths of beam up to a maximum of 2 inches, since table 2 shows that with this span length there is the least variation in modulus of elasticity with changes in depth of beam. When it is necessary to vary both the span length and depth of beam, the following dimensions are suggested as likely to result in the least variation in the modulus of elasticity:

Depth:	*Minimum span (inches)*
Less than 1.3 inches	6
1.3 to 1.7 inches	8
Over 1.7 inches	10

3. The rate at which unit fiber stress is developed should be constant and this may be accomplished by applying the load at the rate determined by the formula

$$R=\frac{Cbd^2}{l}$$

in which

R=rate of loading in pounds per minute;
C=constant; and
l, b, and d are the same as in the previous formulas in this report. Two hundred and twenty is suggested as a suitable value for C. The rate of loading determined with $C=220$ will allow a wide range in the size of the specimen that can be tested on the machine that has been described and the load will be applied at a slow enough rate so that deflection measurements every 10 seconds will give a good load-deformation curve.

(*Continued from p. 78*)

strength, but asphalt A was much more susceptible to temperature and specimens made with both grades of asphalt A had lower stability at 140° F. than specimens made with the corresponding grades of asphalt M.

Asphalts M and C had about the same susceptibility to temperature. Table 4 shows asphalt C to have had less bonding strength than asphalt M, and specimens made with both grades of asphalt C had less stability at 140° F. than specimens made with the corresponding grades of asphalt M (tables 1 and 3).

A stability test using a standard aggregate made at a temperature at which the contained asphalt had a definite penetration might prove to be of value in testing the bonding strength of asphalts for use in road construction.

Because the stability of mixtures containing asphalts of the same type, that is, having the same susceptibility to temperature and the same bonding strength, depends only upon the consistency of the asphalt at the temperature of the stability test, it is not necessary to make tests with all grades of asphalt in order to determine the effect of using different grades of asphalt upon stability. By determining the stability at several temperatures of a mixture containing one grade of asphalt whose penetration has been determined at several temperatures, temperature-penetration and penetration-stability curves can be drawn. For similar types of asphalt the penetration at any temperature of any grade of asphalt can be approximated from the temperature-penetration curve for one grade of asphalt. The stability of the mixture can then be estimated from the penetration-stability curve.

SUMMARY

In these tests on mixtures of constant composition except for the type and consistency of the asphalt it was found that:

1. There were greater differences in the stability of mixtures made with the same grade of asphalts from different producers than there were in the stability of mixtures made with the 50- and 100-penetration grades of asphalt from the same producer. There was as much as 45 percent variation in the stability at 140° F. of the mixtures containing the various 50-penetration asphalts. For materials from the same producer, doubling the penetration of the asphalt reduced the stability of the mixture by from 10 to 30 percent.

2. There is some characteristic of an asphalt which caused mixtures made with various types of asphalt to vary in stability even when the asphalts were of the same consistency when the mixtures were tested for stability. This quality, for lack of a better term, is here called bonding strength.

3. For asphalts of the same type, that is, having approximately the same susceptibility to temperature and the same consistency at their softening point, the stability of the mixtures depended upon the consistency of the asphalt at the temperature of the stability test.

MOTOR-FUEL CONSUMPTION-1939

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

TABLE G-2, 1939
ISSUED MAY 1940

STATE	TAX RATE PER GALLON ON DECEMBER 31	GROSS AMOUNT REPORTED 2/	AMOUNT EXEMPTED FROM PAYMENT OF TAX 3/	GROSS AMOUNT ASSESSED FOR TAXATION	AMOUNT SUBJECT TO REFUND OF ENTIRE TAX	NET AMOUNT TAXED				AMOUNT TAXED AT PREVAILING RATE DURING 1938	INCREASE OR DECREASE DURING 1939		STATE
						TOTAL	AT PREVAILING RATE	AT OTHER RATES			AMOUNT	PERCENT	
								RATE PER GALLON	AMOUNT				
	CENTS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	CENTS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS		
ALABAMA	6	241,375	-	241,375	-	241,375	241,375	-	-	226,838	14,537	6.4	ALABAMA
ARIZONA	5	107,475	5,167	102,308	12,369	89,939	89,939	-	-	84,534	5,405	6.4	ARIZONA
ARKANSAS	6.5	181,922	7,312	174,610	-	174,610	155,709	(4/)	18,901	143,479	12,230	8.5	ARKANSAS
CALIFORNIA	3	1,852,859	27,148	1,825,711	151,931	1,673,780	1,673,780	-	-	1,571,928	101,852	6.5	CALIFORNIA
COLORADO	4	237,669	8,508	229,161	33,014	196,147	196,147	-	-	187,944	8,203	4.4	COLORADO
CONNECTICUT	3	351,315	8,447	342,868	7,722	335,146	335,146	-	-	312,710	22,436	7.2	CONNECTICUT
DELAWARE	4	59,316	1,269	58,047	3,637	54,410	54,410	-	-	52,490	1,920	3.7	DELAWARE
FLORIDA	7	363,710	13,621	350,089	-	350,089	350,089	-	-	326,838	23,251	7.1	FLORIDA
GEORGIA	6	362,403	9,541	352,862	-	352,862	352,862	-	-	328,921	23,941	7.3	GEORGIA
IDAHO	5/ 5.1	100,802	3,635	97,167	-	97,167	6/ 88,444	(7/)	8,723	81,888	6,556	8.0	IDAHO
ILLINOIS	3	1,448,697	-	1,448,697	112,464	1,336,233	1,336,233	-	-	1,256,016	80,217	6.4	ILLINOIS
INDIANA	4	651,249	2,016	649,233	50,499	598,734	598,734	-	-	562,879	35,855	6.4	INDIANA
IOWA	3	550,333	-	550,333	81,231	469,102	469,102	-	-	445,906	23,196	5.2	IOWA
KANSAS	3	467,296	132,719	334,577	-	334,577	334,577	-	-	337,527	-2,950	-0.9	KANSAS
KENTUCKY	5	275,107	-	275,107	-	275,107	275,107	-	-	256,516	18,591	7.2	KENTUCKY
LOUISIANA	7	261,304	6,228	255,076	5	255,071	247,419	2	8/ 7,652	234,941	12,478	5.3	LOUISIANA
MAINE	4	150,136	831	149,305	-	149,305	141,850	1	9/ 7,455	137,406	4,444	3.2	MAINE
MARYLAND	4	291,666	4,106	287,560	19,633	267,927	265,548	3	10/ 2,379	246,433	19,115	7.8	MARYLAND
MASSACHUSETTS	3	721,112	2,829	718,283	34,550	683,733	683,733	-	-	662,254	21,479	3.2	MASSACHUSETTS
MICHIGAN	3	1,146,327	99,359	1,046,968	52,381	994,587	994,058	1.5	11/ 529	928,920	65,138	7.0	MICHIGAN
MINNESOTA	4	559,186	25,686	533,500	63,922	469,578	469,578	-	-	447,444	22,134	4.9	MINNESOTA
MISSISSIPPI	6	205,963	9,032	196,931	-	196,931	188,426	1	12/ 8,505	171,044	17,382	10.2	MISSISSIPPI
MISSOURI	2	653,611	-	653,611	32,820	620,791	620,791	-	-	581,086	39,705	6.8	MISSOURI
MONTANA	5	128,980	8,497	120,483	21,564	98,919	98,919	-	-	89,450	9,469	10.6	MONTANA
NEBRASKA	5	242,721	10,515	232,206	6	232,200	232,119	1	13/ 81	223,309	8,810	3.9	NEBRASKA
NEVADA	4	39,258	2,494	36,764	2,370	34,394	33,618	5	14/ 776	29,958	3,660	12.2	NEVADA
NEW HAMPSHIRE	4	91,831	-	91,831	3,383	88,448	88,448	-	-	82,714	5,734	6.9	NEW HAMPSHIRE
NEW JERSEY	3	849,139	3,055	846,084	72,738	773,346	773,346	-	-	742,435	30,911	4.2	NEW JERSEY
NEW MEXICO	5	102,064	6,349	95,715	9,270	86,445	86,374	7.5	15/ 71	81,521	4,853	6.0	NEW MEXICO
NEW YORK	4	1,900,715	69,381	1,831,334	63,046	1,768,288	1,768,288	-	-	1,684,672	83,616	5.0	NEW YORK
NORTH CAROLINA	6	430,291	7,597	422,694	-	422,694	410,340	1	12/ 12,354	385,834	24,506	6.4	NORTH CAROLINA
NORTH DAKOTA	16/ 4	130,238	27,652	102,586	19,892	82,694	17/ 82,694	-	-	85,775	-3,081	-3.6	NORTH DAKOTA
OHIO 18/	4	1,371,266	68,887	1,302,379	11,494	1,290,885	1,231,223	1	9/ 59,662	1,157,015	74,208	6.4	OHIO 18/
OKLAHOMA	4	421,496	39,967	381,529	15,794	365,735	365,735	-	-	350,190	15,545	4.4	OKLAHOMA
OREGON	5	245,746	5,298	240,448	26,912	213,536	212,609	1	19/ 927	197,797	14,812	7.5	OREGON
PENNSYLVANIA	4	1,482,428	6,351	1,476,077	-	1,476,077	1,476,077	-	-	1,397,068	79,009	5.7	PENNSYLVANIA
RHODE ISLAND	3	129,878	1,198	128,680	1,985	126,695	126,695	-	-	116,874	9,821	8.4	RHODE ISLAND
SOUTH CAROLINA	6	210,980	-	210,980	4,027	206,953	206,953	-	-	188,783	18,170	9.6	SOUTH CAROLINA
SOUTH DAKOTA	4	136,628	4,934	131,694	29,722	101,972	101,972	-	-	99,668	2,304	2.3	SOUTH DAKOTA
TENNESSEE	7	288,738	17,025	271,713	1,491	270,222	270,222	-	-	264,163	6,059	2.3	TENNESSEE
TEXAS	4	1,337,584	16,626	1,320,958	180,516	1,140,442	1,140,442	-	-	1,075,851	64,591	6.0	TEXAS
UTAH	4	99,746	5,397	94,349	-	94,349	94,349	-	-	87,850	6,499	7.4	UTAH
VERMONT	4	68,009	872	67,137	-	67,137	67,137	-	-	63,300	3,837	6.1	VERMONT
VIRGINIA	5	382,173	-	382,173	23,532	358,641	358,541	3	20/ 100	334,327	24,214	7.2	VIRGINIA
WASHINGTON	5	354,142	8,090	346,052	25,111	320,941	320,941	-	-	309,697	11,244	3.6	WASHINGTON
WEST VIRGINIA	5	215,100	-	215,100	3,026	212,074	212,074	-	-	188,915	23,159	12.3	WEST VIRGINIA
WISCONSIN	4	566,693	17,101	549,592	41,816	507,776	507,776	-	-	484,812	22,964	4.7	WISCONSIN
WYOMING	4	68,014	2,341	65,673	-	65,673	65,673	-	-	63,376	2,297	3.6	WYOMING
DISTRICT OF COLUMBIA	2	150,365	6,523	143,842	1,066	142,776	142,776	-	-	133,325	9,451	7.1	DISTRICT OF COLUMBIA
TOTAL	21/ 3.96	22,685,056	703,604	21,981,452	1,214,939	20,766,513	20,638,398	-	128,115	19,504,621	1,133,777	5.8	TOTAL

1/ AN ANALYSIS OF MOTOR-FUEL USAGE WILL BE GIVEN IN TABLE G-21, TO BE PUBLISHED LATER.

2/ EXPORT SALES AND OTHER AMOUNTS NOT REPRESENTING CONSUMPTION IN STATE HAVE BEEN ELIMINATED AS FAR AS POSSIBLE. IN CASES WHERE STATES FAILED TO REPORT AMOUNTS EXEMPTED FROM TAXATION, THE GROSS AMOUNT TAXED IS SHOWN IN THIS COLUMN.

3/ INCLUDES ALLOWANCES FOR EVAPORATION AND OTHER LOSSES, FEDERAL USE, OTHER PUBLIC USE, AND NONHIGHWAY USE, WHERE INITIAL EXEMPTIONS RATHER THAN REFUNDS ARE MADE.

4/ WITHIN 300 FEET OF BORDER, TAX IS REDUCED TO THAT OF ADJACENT STATE. GALLONS TAXED AT 2 CENTS, 4,807,000; AT 4 CENTS, 14,094,000.

5/ RATE CHANGED FROM 5 CENTS TO 5.1 CENTS MARCH 11.

6/ GALLONS TAXED AT 5 CENTS, 3,003,000; AT 5.1 CENTS, 85,441,000.

7/ AVIATION FUEL TAXED AT 2.5 CENTS, 180,000 GALLONS; MOTOR FUEL TAXED AT 0.1 CENT (NONHIGHWAY USE REFUNDED 5 CENTS), 8,543,000 GALLONS.

8/ REPRESENTS EVAPORATION OR LOSS ALLOWANCE UNDER 5-CENT TAX NOT ALLOWED UNDER ADDITIONAL 2-CENT TAX, WHICH IS ADMINISTERED UNDER A SEPARATE LAW.

9/ THREE CENTS PER GALLON REFUNDED ON NONHIGHWAY USES.

10/ ONE CENT PER GALLON REFUNDED ON MOTOR FUEL USED IN VEHICLES LICENSED TO OPERATE EXCLUSIVELY IN CITIES.

11/ ONE AND ONE-HALF CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN INTERSTATE AVIATION.

12/ FIVE CENTS PER GALLON REFUNDED ON NONHIGHWAY USES.

13/ AVIATION FUEL USED IN FLYING INSTRUCTION.

14/ DIESEL FUEL TAXED AT 5 CENTS PER GALLON EFFECTIVE JULY 1.

15/ DIESEL FUEL TAXED AT 7.5 CENTS PRIOR TO JULY 1. TAXED AT REGULAR RATE THEREAFTER.

16/ RATE CHANGED FROM 3 CENTS TO 4 CENTS JULY 1.

17/ GALLONS TAXED AT 3 CENTS, 35,693,000; AT 4 CENTS, 47,001,000.

18/ AMOUNTS GIVEN DO NOT INCLUDE 67,765,000 GALLONS OF LIQUID FUEL (KEROSENE, FUEL OIL, ETC.) TAXED AT 1 CENT PER GALLON BUT NOT SUBJECT TO THE 3-CENT TAX ON MOTOR-VEHICLE FUEL.

19/ FOUR CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN AVIATION.

20/ TWO CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN INTRASTATE AVIATION.

21/ WEIGHTED AVERAGE RATE.

STATE MOTOR-FUEL TAX RECEIPTS - 1939

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

TABLE G-1, 1939
ISSUED MAY 1940

STATE	TAX RATE PER GALLON ON DECEMBER 31	RECEIPTS FROM TAXATION OF MOTOR FUEL					OTHER RECEIPTS IN CONNECTION WITH MOTOR-FUEL TAX 2/					NET TOTAL RECEIPTS	LESS TAX ON AVIATION GASOLINE	ADJUSTED NET TOTAL RECEIPTS	STATE
		GROSS TAX COLLECTIONS	DEDUCTIONS BY DISTRIBUTORS FOR EXPENSES 1/	GROSS RECEIPTS BY STATE	REFUNDS PAID	NET RECEIPTS BY STATE	DISTRIBUTORS' AND DEALERS' LICENSES	INSPECTION FEES 3/	FINES AND PENALTIES	MISCELLANEOUS RECEIPTS 4/	TOTAL				
	CENTS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	6	14,415	-	14,415	-	14,415	-	61	-	-	61	14,476	-	14,476	ALABAMA
ARIZONA	5	5,189	-	5,189	744	4,445	*	-	1	-	1	4,446	-	4,446	ARIZONA
ARKANSAS	6.5	10,647	-	10,647	-	10,647	-	95	*	-	95	10,742	-	10,742	ARKANSAS
CALIFORNIA	3	54,353	-	54,353	4,558	49,795	15	-	-	1	16	49,811	-	49,811	CALIFORNIA
COLORADO	4	8,961	-	8,961	1,210	7,751	-	-	-	-	-	7,751	-	7,751	COLORADO
CONNECTICUT	3	10,193	51	10,142	232	9,910	51	-	-	-	51	9,961	-	9,961	CONNECTICUT
DELAWARE	4	2,288	-	2,288	145	2,143	3	-	-	-	3	2,146	-	2,146	DELAWARE
FLORIDA	7	24,323	-	24,323	-	24,323	34	410	-	-	444	24,767	-	24,767	FLORIDA
GEORGIA	6	21,264	210	21,054	-	21,054	34	-	-	-	34	21,088	-	21,088	GEORGIA
IDAHO	5/ 5.1	4,892	-	4,892	425	4,467	*	-	-	1	1	4,468	4	4,464	IDAHO
ILLINOIS	3	43,159	863	42,296	3,296	39,000	-	399	1	-	400	39,400	-	39,400	ILLINOIS
INDIANA	4	25,845	-	25,845	2,020	23,825	*	492	-	1	493	24,318	-	24,318	INDIANA
IOWA	3	16,432	-	16,432	2,461	13,971	62	-	-	-	62	14,033	-	14,033	IOWA
KANSAS	3	10,023	-	10,023	-	10,023	15	106	-	35	156	10,179	-	10,179	KANSAS
KENTUCKY	5	13,974	140	13,834	-	13,834	-	-	2	-	2	13,836	-	13,836	KENTUCKY
LOUISIANA	7	17,381	-	17,381	*	17,381	-	82	*	-	6/ 82	17,463	-	17,463	LOUISIANA
MAINE	4	5,946	-	5,946	224	5,722	-	-	*	-	*	5,722	4	5,718	MAINE
MARYLAND	4	11,447	-	11,447	809	10,638	-	-	-	-	-	10,638	-	10,638	MARYLAND
MASSACHUSETTS	3	21,548	-	21,548	1,036	20,512	-	-	*	-	*	20,512	-	20,512	MASSACHUSETTS
MICHIGAN	3	31,409	-	31,409	1,579	29,830	5	-	7	-	12	29,842	48	29,794	MICHIGAN
MINNESOTA	4	21,216	-	21,216	2,570	18,646	1	160	2	4	167	18,813	-	18,813	MINNESOTA
MISSISSIPPI 7/	6	11,735	-	11,735	426	11,309	-	-	-	-	-	11,309	-	11,309	MISSISSIPPI 7/
MISSOURI	2	12,757	-	12,757	661	12,096	-	131	7	-	138	12,234	-	12,234	MISSOURI
MONTANA	5	5,887	-	5,887	1,087	4,800	-	7	-	-	7	4,807	-	4,807	MONTANA
NEBRASKA	5	11,828	89	11,739	258	11,481	9	112	-	33	154	11,635	50	11,585	NEBRASKA
NEVADA	4	1,459	28	1,431	95	1,336	*	23	*	-	23	1,359	-	1,359	NEVADA
NEW HAMPSHIRE	4	3,643	-	3,643	135	3,508	-	-	*	-	*	3,508	-	3,508	NEW HAMPSHIRE
NEW JERSEY	3	25,162	-	25,162	2,156	23,006	61	-	1	-	62	23,068	-	23,068	NEW JERSEY
NEW MEXICO	5	4,758	-	4,758	463	4,295	25	-	-	-	25	4,320	-	4,320	NEW MEXICO
NEW YORK	4	72,866	728	72,138	2,445	69,693	64	-	-	*	64	69,757	-	69,757	NEW YORK
NORTH CAROLINA	6	25,149	-	25,149	617	24,532	-	1,017	-	7	1,024	25,556	-	25,556	NORTH CAROLINA
NORTH DAKOTA	8/ 4	3,485	52	3,433	785	2,648	1	66	-	21	88	2,736	-	2,736	NORTH DAKOTA
OHIO	4	9/ 52,656	-	52,656	2,190	50,466	-	-	-	-	-	50,466	-	50,466	OHIO
OKLAHOMA	4	15,258	305	14,953	747	14,206	-	284	5	-	289	14,495	-	14,495	OKLAHOMA
OREGON	5	12,010	-	12,010	1,419	10,591	-	-	*	-	*	10,591	9	10,582	OREGON
PENNSYLVANIA	4	60,256	672	59,584	-	59,584	-	-	6	-	6	59,590	-	59,590	PENNSYLVANIA
RHODE ISLAND	3	4,257	-	4,257	230	4,027	4	-	-	-	4	4,031	-	4,031	RHODE ISLAND
SOUTH CAROLINA	6	12,562	-	12,562	265	12,297	-	250	-	-	250	12,547	31	12,516	SOUTH CAROLINA
SOUTH DAKOTA	4	5,247	107	5,140	1,184	3,956	-	67	-	-	67	4,023	11	4,012	SOUTH DAKOTA
TENNESSEE	7	18,853	-	18,853	97	18,756	-	1,018	-	58	1,076	19,832	105	19,727	TENNESSEE
TEXAS	4	52,939	530	52,409	7,220	45,189	-	-	-	19	19	45,208	-	45,208	TEXAS
UTAH	4	3,793	58	3,735	-	3,735	1	-	*	*	1	3,736	48	3,688	UTAH
VERMONT	4	2,679	-	2,679	-	2,679	-	-	*	-	*	2,679	-	2,679	VERMONT
VIRGINIA	5	19,005	-	19,005	1,174	17,831	*	-	1	-	1	17,832	3	17,829	VIRGINIA
WASHINGTON	5	17,227	-	17,227	1,233	15,994	3	-	-	9	12	16,006	-	16,006	WASHINGTON
WEST VIRGINIA	5	10,560	-	10,560	151	10,409	14	-	-	-	14	10,423	-	10,423	WEST VIRGINIA
WISCONSIN	4	21,949	-	21,949	1,678	20,271	-	167	-	-	167	20,438	-	20,438	WISCONSIN
WYOMING	4	2,622	-	2,622	-	2,622	2	-	-	-	2	2,624	44	2,580	WYOMING
DISTRICT OF COLUMBIA	2	2,805	-	2,805	21	2,784	7	-	-	-	7	2,791	-	2,791	DISTRICT OF COLUMBIA
TOTAL	10/ 3.96	868,312	3,833	864,479	48,046	816,433	411	4,947	33	189	5,580	822,013	357	821,656	TOTAL

1/ THE STATES FOR WHICH AMOUNTS ARE SHOWN MAKE ALLOWANCES TO DISTRIBUTORS FOR EXPENSE OF COLLECTING THE TAX. IN CONNECTICUT, KENTUCKY, SOUTH DAKOTA, AND UTAH ALLOWANCES OF 1, 2 1/4, 4, AND 3 PERCENT, RESPECTIVELY, OF THE TAX OTHERWISE DUE ARE MADE IN CONSIDERATION OF BOTH EXPENSE OF COLLECTION AND GALLONAGE LOSSES IN HANDLING. IN THESE STATES THE ALLOWANCES FOR EXPENSES ONLY HAVE BEEN ESTIMATED AS ½, 1, 2, AND 1½ PERCENT, RESPECTIVELY.

2/ STARS INDICATE AMOUNTS LESS THAN $500.

3/ FEES FOR INSPECTION OF MOTOR-VEHICLE FUEL. WHEREVER POSSIBLE, FEES FOR INSPECTION OF KEROSENE AND OTHER NON-MOTOR-VEHICLE FUELS HAVE BEEN ELIMINATED.

4/ INCLUDES FEES FOR MOTOR-FUEL CARRIER PERMITS, REFUND OR EXEMPTION PERMITS, AND MISCELLANEOUS UNCLASSIFIED RECEIPTS.

5/ RATE CHANGED FROM 5 CENTS TO 5.1 CENTS MARCH 11.

6/ RECEIPTS FROM TAX ON LUBRICATING OIL, $853,000, NOT INCLUDED IN THIS TABLE.

7/ SPECIAL COUNTY TAXES OF 3 CENTS PER GALLON IN HANCOCK COUNTY AND 2 CENTS PER GALLON IN HARRISON COUNTY, AMOUNTING TO $175,000 IN 1939, ARE IMPOSED FOR SEAWALL PROTECTION AND ARE NOT INCLUDED IN THIS TABLE.

8/ RATE CHANGED FROM 3 CENTS TO 4 CENTS JULY 1.

9/ OHIO IMPOSES A 3-CENT TAX ON MOTOR-VEHICLE FUEL AND A 1-CENT TAX ON ALL LIQUID FUELS. THE RECEIPTS FROM THE 1-CENT TAX APPLICABLE TO NON-MOTOR-VEHICLE FUELS (KEROSENE, FUEL OIL, ETC.) WERE $641,000. THESE RECEIPTS HAVE BEEN ELIMINATED FROM THE TOTAL GIVEN, WHICH REPRESENTS A 4-CENT TAX ON MOTOR-VEHICLE FUEL.

10/ WEIGHTED AVERAGE RATE.

STATE MOTOR-VEHICLE REGISTRATIONS-1939

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

TABLE MV-1, 1939
ISSUED MAY 1940

STATE	TOTAL REGISTERED MOTOR VEHICLES, PUBLIC, PRIVATE AND COMMERCIAL 2/	PRIVATE AND COMMERCIAL VEHICLES 3/							PUBLICLY OWNED VEHICLES						DEALERS' REGISTRATIONS AND PLATES 8/		1938 TOTAL REGISTERED MOTOR VEHICLES, PRIVATE AND COMMERCIAL ONLY	YEAR'S CHANGE IN PRIVATE AND COMMERCIAL MOTOR-VEHICLE REGISTRATIONS		STATE
		MOTOR VEHICLES					OTHER REGISTERED VEHICLES		FEDERAL 6/			STATE, COUNTY, AND MUNICIPAL 7/								
			PASSENGER MOTOR VEHICLES																	
		TOTAL MOTOR VEHICLES	TOTAL	AUTOMOBILES (INCLUDING TAXICABS)	MOTOR BUSSES 4/	MOTOR TRUCKS, TRACTOR TRUCKS, ETC.	TRAILERS AND SEMI-TRAILERS 5/	MOTOR-CYCLES	MOTOR VEHICLES	TRAILERS AND SEMI-TRAILERS	MOTOR-CYCLES	MOTOR VEHICLES	TRAILERS AND SEMI-TRAILERS	MOTOR-CYCLES	REGULAR REGIS-TRATIONS	EXTRA SETS OF PLATES		INCREASE OR DECREASE	PER-CENT-AGE CHANGE	
ALABAMA	331,742	325,472	266,582	262,847	3,735	58,890	4,574	1,013	2,060	52	6	4,210	-	-	3,150	-	301,990	23,482	7.8	ALABAMA
ARIZONA	136,037	131,355	107,272	106,924	348	24,083	4,628	440	2,533	88	3	2,149	178	18	285	1,372	128,791	2,564	2.0	ARIZONA
ARKANSAS	245,707	241,425	181,052	180,555	497	60,373	10,707	549	2,143	37	5	2,139	23	25	561	-	220,391	21,034	9.5	ARKANSAS
CALIFORNIA	2,642,006	2,606,590	2,298,608	2,298,608	(9/)	307,982	155,316	13,271	8,818	332	76	26,598	1,766	1,189	4,591	-	2,510,867	95,723	3.8	CALIFORNIA
COLORADO	345,884	343,487	286,913	285,787	1,126	56,574	1,574	1,355	2,397	29	8	-	-	-	3,858	-	332,774	10,713	3.2	COLORADO
CONNECTICUT	465,346	460,870	387,299	386,131	1,168	73,571	5,951	2,008	700	20	14	3,776	171	308	2,957	8,176	440,335	20,535	4.7	CONNECTICUT
DELAWARE	69,109	67,992	56,744	56,744	(9/)	11,248	3,047	280	372	7	-	745	42	-	747	-	64,078	3,914	6.1	DELAWARE
FLORIDA	458,615	451,357	378,116	376,707	1,409	73,241	18,793	1,630	1,851	36	18	5,407	343	161	2,729	-	423,021	28,336	6.7	FLORIDA
GEORGIA	477,713	469,440	387,489	384,773	2,716	81,951	13,617	1,473	3,475	113	32	4,798	77	120	420	2,166	432,360	37,080	8.6	GEORGIA
IDAHO	156,820	153,226	121,714	121,589	125	31,512	21,513	612	1,913	47	1	1,681	108	11	388	-	137,851	15,375	11.2	IDAHO
ILLINOIS	1,863,486	1,849,623	1,624,031	1,624,031	(9/)	225,592	24,966	6,525	3,909	136	29	9,954	330	702	4,363	-	1,780,865	68,758	3.9	ILLINOIS
INDIANA	969,593	960,842	824,196	822,942	1,254	136,646	71,584	5,192	2,137	141	21	6,614	-	192	2,587	-	922,788	38,054	4.1	INDIANA
IOWA	774,227	766,412	671,858	671,858	(9/)	94,554	10/ 98,292	2,753	1,605	38	12	6,210	410	73	1,941	4,497	740,021	26,391	3.6	IOWA
KANSAS	575,980	574,472	475,856	475,464	392	98,616	5,795	1,251	1,508	78	15	-	-	-	2,068	-	573,985	487	0.1	KANSAS
KENTUCKY	444,218	437,188	367,903	367,236	667	69,285	(11/)	1,181	2,582	88	98	4,448	-	-	782	-	414,207	22,981	5.5	KENTUCKY
LOUISIANA	346,820	340,091	263,188	260,145	3,043	76,903	13,673	1,264	1,916	59	11	4,813	355	69	326	-	326,199	13,892	4.3	LOUISIANA
MAINE	203,793	200,867	157,605	157,426	179	43,262	12/ 10,658	796	640	22	10	2,286	181	38	743	-	196,690	4,177	2.1	MAINE
MARYLAND	430,095	425,064	365,376	364,323	1,053	59,688	4,641	1,605	2,808	86	20	2,223	-	-	8,679	-	395,347	29,717	7.5	MARYLAND
MASSACHUSETTS	874,932	871,936	765,656	760,838	4,818	106,280	14,585	712	2,996	84	38	(13/)	-	200	3,253	18,904	843,789	28,147	3.3	MASSACHUSETTS
MICHIGAN	1,475,616	1,472,311	1,326,808	14/ 1,325,918	890	145,503	154,262	4,619	3,305	128	26	-	-	-	1,903	1,903	1,408,835	63,476	4.5	MICHIGAN
MINNESOTA	848,572	840,050	721,473	720,912	561	118,577	42,175	2,324	3,221	120	19	5,301	-	-	1,791	-	821,241	18,809	2.3	MINNESOTA
MISSISSIPPI	248,789	244,086	186,989	185,475	1,514	57,097	4,149	409	1,940	42	1	2,763	-	45	2,823	-	215,195	28,591	15/ 13.4	MISSISSIPPI
MISSOURI	881,946	877,094	735,485	734,894	591	141,609	34,317	2,299	2,606	57	9	2,246	-	9	2,289	3,282	837,118	39,976	4.8	MISSOURI
MONTANA	185,327	180,319	135,839	135,839	(9/)	44,480	4,195	503	2,579	49	1	2,429	55	-	952	-	171,326	8,993	5.2	MONTANA
NEBRASKA	411,464	406,905	343,879	343,563	316	63,026	43,473	1,244	1,238	17	7	3,321	447	52	1,440	1,993	407,330	-425	-0.1	NEBRASKA
NEVADA	42,296	40,771	32,781	32,586	195	7,990	1,361	139	846	15	1	679	33	8	71	275	38,424	2,347	6.1	NEVADA
NEW HAMPSHIRE	131,963	131,302	102,644	102,409	235	28,658	5,708	817	661	33	4	-	-	-	573	-	124,379	6,923	5.6	NEW HAMPSHIRE
NEW JERSEY	1,041,337	1,027,747	894,061	889,081	4,980	133,686	7,679	4,822	3,047	88	27	10,543	-	523	2,470	11,823	1,000,684	27,063	2.7	NEW JERSEY
NEW MEXICO	123,549	120,441	91,953	91,476	477	28,488	2,762	379	1,995	87	3	1,113	-	38	161	67	116,537	3,904	3.4	NEW MEXICO
NEW YORK	2,689,288	2,655,733	2,324,451	2,315,503	8,948	331,282	46,845	11,852	6,315	193	95	27,240	964	1,070	5,289	-	2,584,123	71,610	2.8	NEW YORK
NORTH CAROLINA	587,832	573,497	486,548	485,616	932	86,949	44,882	1,797	2,210	44	2	12,125	-	-	8,705	-	537,242	36,255	6.7	NORTH CAROLINA
NORTH DAKOTA	178,161	176,501	142,523	142,384	139	33,978	984	284	906	19	2	754	-	-	542	376	174,256	2,245	1.3	NORTH DAKOTA
OHIO	1,910,468	1,886,984	1,702,761	1,702,761	(9/)	184,223	134,174	8,935	3,005	150	40	20,479	1,164	406	4,031	20,290	1,870,249	16,735	0.9	OHIO
OKLAHOMA	565,864	556,163	457,991	455,771	2,220	98,172	14,757	1,142	2,644	76	5	7,057	-	-	3,701	-	535,399	20,764	3.9	OKLAHOMA
OREGON	376,736	369,330	306,581	305,943	638	62,749	(9/)	1,693	3,191	149	16	4,215	-	-	656	812	357,321	12,009	3.4	OREGON
PENNSYLVANIA	2,082,862	2,054,787	1,803,366	1,798,032	5,334	251,421	30,738	11,238	3,873	171	44	24,202	480	960	29,917	-	1,976,466	78,321	4.0	PENNSYLVANIA
RHODE ISLAND	177,069	175,068	155,369	154,916	453	19,699	686	730	565	37	7	1,436	24	104	372	-	168,888	6,180	3.7	RHODE ISLAND
SOUTH CAROLINA	321,235	314,628	270,901	268,633	2,268	43,727	5,598	929	2,069	47	4	4,538	-	88	1,002	2,190	287,913	26,715	9.3	SOUTH CAROLINA
SOUTH DAKOTA	192,111	189,485	159,099	158,998	101	30,386	21,821	456	1,410	30	8	1,216	234	-	789	-	180,632	8,853	4.9	SOUTH DAKOTA
TENNESSEE	433,982	423,956	356,903	356,183	720	67,053	(9/)	1,758	2,451	58	2	7,575	-	-	588	-	398,624	25,332	6.4	TENNESSEE
TEXAS	1,641,662	1,618,039	1,282,398	1,281,566	832	335,641	54,514	4,562	6,230	263	47	17,393	1,172	360	3,199	-	1,548,343	69,696	4.5	TEXAS
UTAH	135,935	132,871	111,656	110,980	676	21,215	558	486	2,010	34	13	1,054	20	40	338	966	127,004	5,867	4.6	UTAH
VERMONT	91,407	90,713	81,137	81,041	96	16/ 9,576	2,026	482	694	16	1	-	-	-	612	-	87,402	3,311	3.8	VERMONT
VIRGINIA	469,518	460,416	390,498	390,498	-	69,918	9,877	2,079	3,613	235	57	5,489	61	251	5,553	-	441,462	18,954	4.3	VIRGINIA
WASHINGTON	546,435	535,799	450,305	449,333	972	85,494	20,882	2,171	3,443	171	23	7,193	363	160	1,503	3,311	523,328	12,471	2.4	WASHINGTON
WEST VIRGINIA	292,484	286,167	237,878	237,189	689	48,289	3,382	1,208	1,002	21	2	5,315	164	71	11,178	-	275,691	10,476	3.8	WEST VIRGINIA
WISCONSIN	859,173	848,658	705,751	705,135	616	142,907	5,783	3,419	2,866	115	15	7,649	212	261	2,351	-	840,291	8,367	1.0	WISCONSIN
WYOMING	84,990	82,884	64,954	64,836	118	17,930	10,795	291	1,340	43	20	766	74	-	356	-	80,765	2,119	2.6	WYOMING
DISTRICT OF COLUMBIA	167,426	164,673	150,955	149,745	1,210	13,718	788	642	1,382	31	56	17/ 1,371	77	76	2,451	-	162,863	1,810	1.1	DISTRICT OF COLUMBIA
AT LARGE	2,250	-	-	-	-	-	-	-	2,250	39	12	-	-	-	-	-	-	-	-	AT LARGE
TOTAL	31,009,870	30,615,087	26,201,395	26,142,144	59,251	4,413,692	1,193,085	117,619	121,270	4,071	986	273,513	9,528	7,628	142,034	82,403	29,485,680	1,129,407	3.8	TOTAL

1/ REGISTRATION PERIODS ENDING NOT EARLIER THAN NOVEMBER 30 AND NOT LATER THAN JANUARY 31 ARE CONSIDERED CALENDAR-YEAR PERIODS. IN THOSE STATES WHERE THE REGISTRATION PERIOD IS DEFINITELY REMOVED FROM THE CALENDAR YEAR, REGISTRATION FIGURES WERE OBTAINED FOR THE CALENDAR-YEAR PERIOD.

2/ TOTALS GIVEN IN THIS COLUMN, WHICH INCLUDE PUBLICLY OWNED AS WELL AS PRIVATE AND COMMERCIAL REGISTRATIONS, DO NOT APPEAR IN TABLES FOR PREVIOUS YEARS. PRIVATE AND COMMERCIAL REGISTRATIONS, GIVEN IN THE SECOND COLUMN, ARE COMPARABLE WITH FIRST COLUMN OF TABLE MV-1 FOR PAST YEARS. MOTOR-VEHICLE TOTALS DO NOT INCLUDE TRAILERS, SEMITRAILERS, AND MOTORCYCLES.

3/ WHEREVER POSSIBLE PUBLICLY OWNED VEHICLES AND VEHICLES NOT FOR HIGHWAY USE HAVE BEEN ELIMINATED FROM THESE COLUMNS.

4/ A COMPLETE SEGREGATION OF MOTOR BUSSES FROM OTHER VEHICLES IS NOT AVAILABLE. THE FIGURES GIVEN REPRESENT COMMON-CARRIER BUSSES IN MOST CASES, ALTHOUGH IN SOME STATES CONTRACT BUSSES AND CONTRACT SCHOOL BUSSES ARE INCLUDED. IN SOME CASES CITY BUSSES ARE NOT INCLUDED. WHERE NO BUSSES ARE TABULATED, THEY ARE INCLUDED WITH AUTOMOBILES, UNLESS OTHERWISE NOTED.

5/ FIGURES FOR TRAILERS AND SEMITRAILERS ARE AS REPORTED. APPARENT INCONSISTENCIES ARE DUE TO THE FACT THAT SOME STATES REQUIRE THE REGISTRATION OF TOURIST TRAILERS, LIGHT WORK TRAILERS, AND SIMILAR VEHICLES, WHEREAS OTHER STATES REGISTER ONLY FREIGHT-CARRYING TRAILERS AND SEMITRAILERS.

6/ DATA ON FEDERAL VEHICLES OBTAINED THROUGH AGENCY OF PROCUREMENT DIVISION, DEPARTMENT OF THE TREASURY.

7/ STATE, COUNTY, AND MUNICIPAL VEHICLES ARE INCLUDED WITH PRIVATE AND COMMERCIAL REGISTRATIONS IN COLORADO, KANSAS, MICHIGAN, NEW HAMPSHIRE, AND VERMONT. A SMALL NUMBER OF FEDERAL VEHICLES ARE INCLUDED IN THE FIGURES FOR INDIANA, KENTUCKY, LOUISIANA, MONTANA, NEBRASKA, NEW YORK, PENNSYLVANIA, AND VIRGINIA.

8/ FIGURES INCLUDE NEW-CAR, USED-CAR, AND MOTORCYCLE DEALERS' REGISTRATIONS AND SOME WRECKER AND REPAIRER REGISTRATIONS. DATA ON DEALERS' EXTRA PLATES ARE INCOMPLETE, ALTHOUGH THEY ARE INCLUDED WITH DEALERS' REGISTRATIONS IN SOME STATES.

9/ INCLUDED WITH MOTOR TRUCKS.

10/ INCLUDES 68,985 LIGHT TRAILERS REGISTERED WITHOUT CHARGE.

11/ TRAILERS OF 1,000 POUNDS CAPACITY OR MORE PROHIBITED ON HIGHWAYS, ALTHOUGH PERMITTED IN CITIES UNDER CITY LICENSES. TRACTOR-SEMITRAILERS REGISTERED AS MOTOR TRUCKS. LIGHT TRAILERS PERMITTED BUT NOT REGISTERED.

12/ INCLUDES LIGHT TRAILERS AND COMMERCIAL SEMITRAILERS. COMMERCIAL FULL TRAILERS INCLUDED WITH MOTOR TRUCKS.

13/ APPROXIMATELY 7,500, BUT CANNOT BE SEGREGATED BY TYPE FROM PRIVATE AND COMMERCIAL VEHICLES.

14/ TAXICABS INCLUDED WITH MOTOR TRUCKS.

15/ LARGE INCREASE IS DUE IN PART TO FACT THAT 1938 FIGURES WERE FOR A TEN-MONTH REGISTRATION PERIOD BECAUSE OF CHANGE IN LICENSE YEAR.

16/ TRUCKS UNDER 1,500 POUNDS CAPACITY INCLUDED WITH PASSENGER CARS.

17/ INCLUDES 399 AUTOMOBILES OF THE DIPLOMATIC CORPS.

STATE MOTOR-VEHICLE RECEIPTS-1939

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

TABLE MV-2, 1939
ISSUED MAY 1940

STATE	TOTAL RECEIPTS, REGISTRATION AND OTHER FEES	MOTOR-VEHICLE REGISTRATION FEES						REGISTRATION FEES, OTHER VEHICLES		TOTAL REGISTRATION FEES, ALL VEHICLES	MISCELLANEOUS RECEIPTS										STATE
		TOTAL 2/	PASSENGER MOTOR VEHICLES			MOTOR TRUCKS, TRACTOR TRUCKS, ETC.															
			TOTAL	AUTOMOBILES (INCLUDING TAXICABS)	MOTOR BUSSES 3/			TRAILERS AND SEMI-TRAILERS	MOTORCYCLES		TOTAL	DEALERS' LICENSES AND PLATES	OPERATORS' AND CHAUFFEURS' PERMITS	CERTIFICATES OF TITLE	SPECIAL TITLING TAXES 4/	FINES AND PENALTIES	TRANSFER OR REREGISTRATION FEES	OTHER RECEIPTS	ESTIMATED SERVICE CHARGES, LOCAL COLLECTORS 5/	UNCLASSIFIED REFUNDS	
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	4,947	4,186	3,160	3,088	72	1,026	52	4	4,242	705	3	524	-	-	168	-	10	-	-	ALABAMA
ARIZONA	1,134	826	397	371	26	429	76	2	904	230	5	25	70	-	5	13	112	-	-	ARIZONA
ARKANSAS	3,140	2,684	1,741	1,665	76	943	171	1	2,856	284	87	183	-	-	3	8	3	-	-	ARKANSAS
CALIFORNIA 6/	24,646	21,503	16,830	16,830	(7/)	4,623	1,064	66	22,633	2,013	83	111	-	-	181	1,378	260	-	-	CALIFORNIA 6/
COLORADO	2,879	2,109	1,631	1,631	-	8/ 478	35	2	2,146	733	29	344	173	-	38	81	68	-	-	COLORADO
CONNECTICUT	6,738	4,456	2,947	2,819	128	1,509	21	6	4,483	2,255	109	1,752	-	-	147	128	125	-	-6	CONNECTICUT
DELAWARE	1,313	943	645	645	(7/)	298	35	1	979	334	7	142	40	-	104	17	24	-	-	DELAWARE
FLORIDA	7,096	6,196	4,310	4,115	195	1,886	314	8	6,518	578	23	210	213	-	-	1	13	118	-	FLORIDA
GEORGIA	2,620	1,612	1,075	958	117	537	218	2	1,832	788	13	734	-	-	-	9	32	-	-	GEORGIA
IDAHO	1,399	1,139	618	609	9	521	64	2	1,205	194	23	129	32	-	-	-	10	-	-	IDAHO
ILLINOIS	24,352	21,152	14,706	14,706	(7/)	6,446	199	17	21,368	2,984	94	1,668	512	-	2	505	232	-	-29	ILLINOIS
INDIANA	10,001	8,020	6,222	6,132	90	1,798	370	10	8,400	1,601	49	674	324	-	32	342	55	261	-136	INDIANA
IOWA	12,624	11,674	8,818	8,818	(7/)	2,856	106	10	11,790	834	62	635	-	-	-	215	12	-	-90	IOWA
KANSAS	4,363	3,587	2,723	2,703	20	864	79	5	3,671	692	31	372	127	-	113	24	25	-	-	KANSAS
KENTUCKY	4,960	3,088	1,855	1,744	111	1,233	(9/)	2	3,090	1,870	23	528	-	1,099	4	202	14	-	-	KENTUCKY
LOUISIANA	5,309	4,524	3,178	3,074	104	1,346	714	5	5,243	66	11	110	-	-	-	-	5	-	-60	LOUISIANA
MAINE	3,788	2,857	1,990	1,972	18	867	10/ 70	4	2,931	857	59	555	38	-	24	139	42	-	-	MAINE
MARYLAND	5,538	3,802	3,161	2,985	176	641	136	6	3,944	1,594	45	312	196	644	283	125	42	-	-53	MARYLAND
MASSACHUSETTS	7,028	4,304	2,857	2,715	142	1,447	20	1	4,325	2,703	66	2,629	-	-	2	-	70	-	-64	MASSACHUSETTS
MICHIGAN	22,119	18,685	13,221	11/ 13,221	(7/)	5,464	1,136	17	19,838	2,281	91	854	780	-	-	319	237	-	-	MICHIGAN
MINNESOTA	9,884	9,089	7,124	6,975	149	1,965	291	8	9,388	496	58	177	-	-	112	-	149	-	-	MINNESOTA
MISSISSIPPI	2,567	2,522	-	-	-	-	-	-	2,522	45	3	-	-	-	36	-	6	-	-	MISSISSIPPI
MISSOURI	10,226	8,690	7,279	7,279	(7/)	1,411	156	12	8,858	1,368	83	457	455	-	2	306	65	-	-	MISSOURI
MONTANA 12/	1,634	1,233	890	890	(7/)	343	18	1	1,252	382	19	180	66	-	70	41	6	-	-	MONTANA 12/
NEBRASKA	2,683	2,139	1,275	1,250	25	864	59	2	2,200	483	29	298	51	-	-	72	36	-	-3	NEBRASKA
NEVADA	286	262	164	163	1	98	6	1	269	17	2	-	-	-	1	12	2	-	-	NEVADA
NEW HAMPSHIRE	2,962	2,469	-	-	-	-	-	4	2,473	489	24	367	-	-	51	-	47	-	-	NEW HAMPSHIRE
NEW JERSEY	21,312	14,541	10,401	10,104	297	4,140	337	10	14,888	6,424	71	4,023	559	-	234	562	985	-	-10	NEW JERSEY
NEW MEXICO	1,909	1,601	1,025	969	56	576	47	1	1,649	260	10	155	27	-	1	-	67	-	-	NEW MEXICO
NEW YORK	48,432	44,747	32,683	32,683	-	12,064	654	49	45,450	2,982	203	904	-	-	463	1,077	359	-	-24	NEW YORK
NORTH CAROLINA	8,163	7,797	4,608	4,539	69	3,189	(7/)	7	7,804	359	33	130	168	-	67	-	12	-	-51	NORTH CAROLINA
NORTH DAKOTA	1,619	1,421	1,095	1,080	15	326	4	2	1,427	192	15	98	57	19	-	-	3	-	-	NORTH DAKOTA
OHIO	28,625	23,259	15,142	15,142	(7/)	8,117	1,308	42	24,609	4,016	246	776	513	-	60	678	32	1,711	-	OHIO
OKLAHOMA	6,046	4,458	2,731	2,627	104	1,727	218	6	4,682	1,364	45	605	287	-	-	-	130	297	-	OKLAHOMA
OREGON	3,469	2,729	1,568	1,530	38	1,161	(7/)	5	2,734	735	25	473	188	-	24	-	25	-	-	OREGON
PENNSYLVANIA	36,563	28,680	19,955	19,395	560	8,725	458	31	29,169	7,394	386	3,140	2,053	-	12	1,460	376	-	-33	PENNSYLVANIA
RHODE ISLAND	2,929	2,324	1,818	1,774	44	506	3	3	2,330	599	17	441	-	-	-	87	54	-	-	RHODE ISLAND
SOUTH CAROLINA	1,915	1,505	821	821	-	684	126	1	1,632	283	26	199	-	-	27	27	43	-	-39	SOUTH CAROLINA
SOUTH DAKOTA	2,165	1,631	1,394	1,390	4	237	37	1	1,669	496	19	-	26	439	-	-	12	-	-	SOUTH DAKOTA
TENNESSEE	5,443	4,647	-	-	-	-	-	-	4,647	796	(13/)	568	-	-	108	-	121	-	-1	TENNESSEE
TEXAS	21,194	19,641	12,631	12,479	152	7,010	587	19	20,247	947	39	417	-	-	-	334	157	-	-	TEXAS
UTAH	1,176	1,012	531	531	(7/)	481	(7/)	1	1,013	163	13	22	72	-	1	21	34	-	-	UTAH
VERMONT	2,490	2,089	1,472	14/ 1,459	13	14/ 617	33	4	2,126	364	27	289	-	-	-	51	7	-	-10	VERMONT
VIRGINIA	6,847	5,740	4,328	4,328	-	1,412	150	6	5,896	951	61	384	298	-	-	173	37	-	-2	VIRGINIA
WASHINGTON	4,560	2,563	1,420	1,420	-	1,143	252	6	2,821	1,739	14	1,284	146	-	33	87	15	160	-	WASHINGTON
WEST VIRGINIA	5,974	4,734	3,630	3,606	24	1,104	58	5	4,797	1,177	45	268	150	648	-	56	10	-	-	WEST VIRGINIA
WISCONSIN	12,954	12,287	9,376	9,135	241	2,911	379	16	12,682	272	10	35	-	-	-	216	11	-	-	WISCONSIN
WYOMING	622	570	369	369	-	201	37	1	608	14	8	-	-	-	-	5	1	-	-	WYOMING
DISTRICT OF COLUMBIA	1,851	1,270	862	848	14	408	20	3	1,293	558	(15/)	262	83	-	-	35	178	-	-	DISTRICT OF COLUMBIA
PARTIAL TOTALS 16/	-	-	236,727	233,637	3,090	96,632	10,118	418	-	-	-	-	-	-	-	-	-	-	-	PARTIAL TOTALS 16/
FULL TOTALS	412,494	342,997	-	-	-	-	-	-	353,533	58,961	2,444	28,443	7,704	2,849	2,408	8,806	4,371	2,547	-611	FULL TOTALS

1/ RECEIPTS FOR REGISTRATION PERIODS ENDING NOT EARLIER THAN NOVEMBER 30 AND NOT LATER THAN JANUARY 31 ARE CONSIDERED CALENDAR-YEAR RECEIPTS. IN THOSE STATES WHERE THE REGISTRATION PERIOD IS DEFINITELY REMOVED FROM THE CALENDAR YEAR, REGISTRATION RECEIPTS WERE OBTAINED FOR THE CALENDAR-YEAR PERIOD.

2/ SEGREGATION OF REGISTRATION FEES BY TYPE OF VEHICLE WAS NOT AVAILABLE FOR MISSISSIPPI, NEW HAMPSHIRE, AND TENNESSEE. TOTAL MOTOR-VEHICLE REGISTRATION FEES IN THOSE STATES INCLUDE TRAILER AND MOTORCYCLE FEES EXCEPT IN NEW HAMPSHIRE, FOR WHICH MOTORCYCLE FEES WERE REPORTED SEPARATELY. DEALERS' LICENSE FEES IN TENNESSEE AND THE DISTRICT OF COLUMBIA ARE ALSO INCLUDED IN THIS COLUMN.

3/ THE MOTOR-BUS REGISTRATION FEES ARE INCOMPLETE (SEE TABLE MV-1, 1939, NOTE 4). WHERE NO FEES ARE TABULATED, THE FEES OF BUSSES ARE INCLUDED WITH THOSE OF AUTOMOBILES, UNLESS OTHERWISE NOTED.

4/ PROCEEDS OF SPECIAL EXCISE AND PRIVILEGE TAXES ON NEW CAR SALES HAVE BEEN SEGREGATED AND ENTERED IN THIS COLUMN. RECEIPTS FROM A 2 PERCENT MOTOR-VEHICLE EXCISE TAX IN OKLAHOMA, IMPOSED AS A PART OF A GENERAL SALES TAX, ARE NOT INCLUDED IN THIS TABLE. PROCEEDS OF THIS TAX WERE $1,188,000 IN 1939.

5/ IN MANY STATES COUNTY OR LOCAL OFFICERS ARE ALLOWED SERVICE CHARGES FOR ISSUING REGISTRATIONS, OPERATORS' LICENSES, ETC. IN THE MAJORITY OF CASES THESE CHARGES ARE INCLUDED IN REGISTRATION AND OTHER FEES. THE AMOUNTS SHOWN IN THIS COLUMN ARE ESTIMATES OF SERVICE CHARGES COLLECTED AND RETAINED BY LOCAL OFFICIALS AND NOT REPORTED ELSEWHERE IN THE TABLE.

6/ REGISTRATION FEES INCLUDE PROCEEDS OF STATE "VEHICLE LICENSE FEES", $10,994,000, IMPOSED IN ADDITION TO THE REGULAR REGISTRATION FEES OF $11,639,000.

7/ INCLUDED WITH MOTOR-TRUCK FEES.

8/ FEES OF 25,934 LIGHT TRUCKS INCLUDED WITH THOSE OF PASSENGER VEHICLES.

9/ TRAILERS OF 1,000 POUNDS CAPACITY OR MORE PROHIBITED ON HIGHWAYS, ALTHOUGH PERMITTED IN CITIES UNDER CITY LICENSES. TRACTOR-SEMITRAILERS REGISTERED AS MOTOR TRUCKS. LIGHT TRAILERS PERMITTED BUT NOT REGISTERED.

10/ FEES OF LIGHT TRAILERS AND COMMERCIAL SEMITRAILERS ONLY. FEES OF COMMERCIAL FULL TRAILERS INCLUDED WITH THOSE OF MOTOR TRUCKS.

11/ FEES OF TAXICABS INCLUDED WITH THOSE OF MOTOR TRUCKS.

12/ REGISTRATION FEES ARE COLLECTED BY COUNTIES AND STATE DOES NOT MAINTAIN COMPLETE RECORD. FIGURES GIVEN ARE ESTIMATES SUPPLIED BY STATE.

13/ INCLUDED WITH MOTOR-VEHICLE REGISTRATION FEES.

14/ FEES OF TRUCKS UNDER 1,500 POUNDS CAPACITY INCLUDED WITH THOSE OF PASSENGER CARS.

15/ INCLUDED WITH PASSENGER CAR FEES.

16/ TOTALS OF COLUMNS FOR WHICH FULLY CLASSIFIED DATA WERE NOT AVAILABLE FOR ALL STATES.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MAY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 6,784,354	$ 3,305,842	283.7	$ 6,808,072	$ 3,651,252	224.1	$ 1,496,570	$ 745,980	63.1	$ 3,484,641
Arizona	2,393,890	1,665,440	127.7				86,950	46,480	5.2	1,794,775
Arkansas	5,117,999	4,022,344	235.2	2,093,534	1,118,170	91.0	1,053,828	521,267	53.0	1,525,366
California	5,910,428	3,129,390	92.2	7,044,277	3,692,573	123.4	2,318,266	1,178,000	53.5	4,036,664
Colorado	3,925,877	2,114,735	89.3	1,923,528	1,082,242	55.2	643,519	277,474	28.6	3,207,225
Connecticut	1,343,854	666,370	16.1	1,687,715	836,108	14.7	873,077	412,448	6.7	1,261,963
Delaware	938,628	448,701	31.5	1,036,072	518,036	7.0	898,058	421,214	22.7	1,176,866
Florida	3,640,769	1,817,537	41.7	3,678,731	1,833,246	114.7	822,431	406,692	11.4	2,461,417
Georgia	4,805,674	2,265,761	240.0	5,555,867	2,777,933	305.1	3,165,524	1,582,762	92.4	6,586,333
Idaho	2,209,950	1,291,400	114.2	1,178,327	720,678	92.2	492,509	279,607	42.6	2,040,278
Illinois	9,392,358	4,615,898	198.8	8,028,484	4,013,647	164.0	2,350,400	1,174,470	80.4	4,567,571
Indiana	4,647,980	2,237,814	88.9	6,179,453	3,083,320	130.5	2,775,764	1,387,780	48.7	2,379,188
Iowa	4,084,454	1,926,642	208.5	5,180,019	2,380,967	163.3	1,902,586	898,100	57.2	1,872,989
Kansas	4,422,161	2,140,702	231.9	2,753,068	1,373,126	134.5	5,866,205	2,918,157	369.6	4,441,249
Kentucky	3,368,541	1,663,813	112.2	2,623,217	1,310,053	44.9	3,333,502	1,666,751	112.6	3,164,847
Louisiana	1,383,213	688,377	43.4	11,914,421	3,000,572	37.5	2,304,632	1,142,195	59.8	3,196,953
Maine	2,180,657	1,078,652	53.2	716,527	358,263	16.4	920,476	460,238	21.8	861,746
Maryland	2,929,809	1,401,605	39.5	1,940,276	951,820	34.6	2,343,200	798,500	22.0	1,711,366
Massachusetts	3,140,962	1,567,753	25.1	1,885,632	939,300	15.9	2,150,212	1,071,216	18.4	3,029,716
Michigan	4,956,387	2,353,044	115.5	5,139,627	2,481,812	164.3	6,204,305	3,102,153	166.8	1,902,050
Minnesota	5,950,627	2,920,578	389.7	4,731,155	2,347,627	282.9	2,503,347	1,251,080	197.6	4,682,926
Mississippi	6,030,168	2,102,799	261.5	6,310,428	2,762,315	273.0	1,783,160	769,862	111.6	2,525,046
Missouri	4,126,910	2,040,405	163.8	4,501,580	2,239,630	151.8	5,096,484	1,890,960	160.6	5,604,260
Montana	3,393,236	1,912,298	200.2	3,807,767	2,156,592	237.7	1,304,973	665,447	93.3	4,293,771
Nebraska	4,830,793	2,332,962	418.0	5,449,599	2,611,385	585.6	2,323,162	1,158,406	332.1	3,277,363
Nevada	1,185,255	1,001,138	57.8	1,394,584	1,199,549	69.6	156,361	136,314	5.0	1,659,566
New Hampshire	814,656	400,642	27.3	1,130,306	555,210	26.2	1,073,658	421,773	21.6	911,349
New Jersey	1,695,210	836,371	15.5	5,564,998	2,782,269	41.8	172,620	86,310		2,145,319
New Mexico	2,552,381	1,552,866	184.9	1,501,097	931,624	90.3	450,009	271,265	55.1	1,931,204
New York	8,719,032	4,197,566	168.5	13,321,897	6,502,532	200.6	3,459,900	1,441,320	48.0	4,184,990
North Carolina	5,750,955	2,859,046	353.3	5,330,166	2,664,742	244.4	1,418,894	709,425	78.3	2,547,186
North Dakota	291,677	154,462	41.7	2,389,943	1,360,324	159.9	2,187,610	1,168,948	233.2	4,417,093
Ohio	6,184,561	3,032,828	86.8	11,977,902	5,965,517	106.1	4,737,350	2,367,775	46.8	5,748,730
Oklahoma	3,653,106	1,934,468	150.7	2,774,235	1,470,047	72.1	2,896,670	1,512,083	141.0	4,235,272
Oregon	3,033,397	1,815,819	113.1	3,886,202	2,208,762	162.5	281,070	165,158	11.3	1,839,645
Pennsylvania	9,609,944	4,717,728	113.7	9,175,583	4,410,784	88.8	6,279,282	3,102,189	62.4	3,805,406
Rhode Island	672,284	325,167	7.8	1,181,937	590,000	11.6	561,095	280,016	5.8	1,009,730
South Carolina	2,756,188	1,236,308	86.6	2,064,007	988,239	126.5	942,963	441,060	108.8	2,635,202
South Dakota	3,390,132	1,870,120	329.2	3,278,840	1,864,180	393.5	2,407,120	1,431,640	322.3	3,464,881
Tennessee	5,182,969	2,511,833	112.7	2,925,700	1,462,850	66.0	1,620,332	810,166	65.4	4,353,971
Texas	14,794,950	7,269,888	763.4	10,263,939	5,082,124	547.9	1,249,075	597,050	66.4	8,497,102
Utah	2,607,850	1,819,663	122.3	860,730	630,957	50.8	612,700	422,547	38.2	1,199,173
Vermont	738,188	348,926	18.4	1,250,998	625,287	38.6	324,040	155,444	5.2	500,398
Virginia	2,511,751	1,242,524	82.2	3,117,371	1,493,696	71.8	689,674	323,547	17.2	2,431,360
Washington	2,501,685	1,270,615	39.6	3,818,299	2,008,429	59.3	1,183,093	629,800	34.0	1,181,745
West Virginia	2,231,182	1,165,064	53.0	1,809,850	897,710	50.2	2,840,440	1,343,373	79.2	1,871,863
Wisconsin	5,150,897	2,525,683	187.6	5,023,976	2,465,950	153.2	1,171,333	573,890	41.7	3,915,134
Wyoming	1,562,428	953,356	141.3	1,888,836	1,199,453	195.9	65,301	41,223	10.9	1,437,863
District of Columbia	708,480	351,590	5.5	173,224	86,612	1.8	427,200	177,488	3.3	432,435
Hawaii	705,318	308,866	5.9	628,220	315,397	13.7	433,940	215,300	7.7	1,614,698
Puerto Rico	736,989	351,635	14.0	1,306,457	646,785	25.3	138,912	67,545	3.0	869,455
TOTALS	191,651,144	97,765,034	7,104.6	200,206,673	98,649,696	6,532.7	92,793,782	45,119,888	3,743.5	143,927,339

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MAY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,081,969	$ 430,989	48.3	$ 526,612	$ 262,480	18.9	$ 432,650	$ 215,150	35.5	$ 629,332
Arizona	327,954	233,525	35.8	155,799	91,426	20.7	71,045	48,750	5.8	380,652
Arkansas	981,558	827,256	95.9	119,046	78,669	8.7	173,137	86,433	20.1	279,773
California	850,189	466,095	36.1	688,043	374,998	32.8	149,817	77,800	1.0	1,013,038
Colorado	1,115,228	568,303	37.9	54,923	8,092	.6				281,982
Connecticut	172,310	72,417	2.9	287,102	139,108	3.3	80,124	38,902	1.3	206,069
Delaware	84,115	39,067	17.5	69,537	34,768	7.8	106,095	38,808	7.7	270,667
Florida	888,923	436,084	31.3	29,916	14,958	.6	101,240	50,620	1.3	565,420
Georgia	363,706	172,132	44.2	360,339	180,170	40.5	294,117	132,059	19.4	1,287,885
Idaho	523,519	288,111	51.9	61,395	37,525	6.0				295,190
Illinois	1,549,557	691,354	93.5	1,697,985	831,818	74.1	521,700	261,000	21.7	490,107
Indiana	896,572	423,398	77.7	409,170	203,381	29.1	157,770	78,850	9.1	947,730
Iowa	609,288	291,392	160.0	1,710,242	812,860	291.9	857,161	403,569	158.9	552,219
Kansas	177,233	88,600	47.2	592,230	296,115	9.0	57,772	28,886	37.1	1,556,610
Kentucky	1,244,012	348,314	82.8	505,621	196,100	31.9	680,533	193,435	62.0	383,773
Louisiana	924,552	447,982	79.6	297,930	148,910	25.7				451,252
Maine	443,074	210,484	25.4	81,124	39,773	4.0	133,950	61,816	7.2	79,839
Maryland	306,943	144,266	20.2	97,996	48,998	3.5	58,000	28,035	4.2	437,550
Massachusetts	373,212	185,203	9.2	443,272	219,589	10.1	228,890	113,575	3.4	477,496
Michigan	1,307,287	633,561	113.8	1,253,519	630,149	80.5	487,690	243,845	53.2	682,873
Minnesota	960,455	465,774	122.3	379,975	157,400	40.1	356,745	178,373	47.8	1,261,496
Mississippi	359,900	179,919	40.7	1,063,962	520,196	52.6				713,924
Missouri	1,036,115	510,401	149.3	652,998	320,918	75.9	198,914	73,927	32.8	809,931
Montana	831,054	466,715	81.0	516,018	289,630	52.7	153,102	86,588	18.1	781,995
Nebraska	1,102,133	520,075	218.7	548,756	274,219	80.4	265,386	132,693	34.4	393,561
Nevada	164,206	134,299	25.0	197,591	169,920	25.5	54,015	47,090	14.6	181,252
New Hampshire	58,755	29,144	2.3	140,209	66,301	4.2				197,644
New Jersey	329,558	161,550	12.1	585,440	292,720	18.3				540,280
New Mexico	469,280	287,227	42.1	367,851	229,448	26.9				262,447
New York	1,818,792	872,861	91.3	2,062,718	995,315	55.5	417,741	174,171	20.7	612,569
North Carolina	1,068,033	530,556	109.6	860,823	431,828	76.1	265,100	120,364	22.3	316,132
North Dakota	114,601	59,497	8.2	62,121	34,895		156,003	83,616	3.8	1,011,462
Ohio	702,537	347,369	41.8	1,929,662	971,606	73.6	1,240,040	610,020	33.0	903,537
Oklahoma	481,426	249,832	30.1	613,296	325,546	45.6	361,500	166,695	18.8	933,839
Oregon	636,928	361,412	73.6	369,156	203,647	41.3	165,907	96,930	25.2	321,377
Pennsylvania	2,134,408	1,022,607	119.9	1,714,364	851,077	57.9	767,145	382,810	16.8	261,397
Rhode Island	93,827	46,890	2.2	230,055	115,001	3.6				95,049
South Carolina	578,846	235,287	56.9	608,940	238,145	51.9	222,119	87,732	65.5	227,321
South Dakota	16,229	8,937	4.1	14,680	9,712					1,280,913
Tennessee	956,289	420,211	32.3	114,396	57,198	7.9	42,142	21,071	3.0	1,000,026
Texas	2,799,513	1,377,896	307.1	1,178,388	577,058	163.9	290,473	137,888	44.0	1,167,078
Utah	312,283	182,505	45.5	31,870	20,500	1.2	75,730	25,000	4.0	269,807
Vermont	138,763	63,935	5.6	280,092	98,529	10.4	133,400	39,000	4.7	48,125
Virginia	680,335	326,094	66.8	537,794	242,820	30.4	91,770	28,192	4.4	360,762
Washington	581,590	301,956	43.1	414,109	217,793	32.7				384,218
West Virginia	207,328	102,063	10.2	311,219	155,084	18.4	113,508	56,754	5.7	441,349
Wisconsin	882,223	438,333	34.5	321,053	160,190	4.7	461,874	225,310	14.4	787,878
Wyoming	470,702	286,620	26.0	386,241	232,715	39.0				186,505
District of Columbia	122,504	60,900	1.5	3,192	1,096		48,200	24,100	.6	60,154
Hawaii	193,033	96,211	6.2	273,208	137,108	8.6				159,806
Puerto Rico	30,220	14,440	2.2	244,135	119,025	12.9	113,278	55,755	3.2	80,408
TOTALS	34,553,067	17,160,049	2,923.4	26,456,113	13,166,527	1,811.9	10,585,783	4,955,612	886.7	27,321,699

U. S. GOVERNMENT PRINTING OFFICE: 1940

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . . Official Inspection of Vehicles. 10 cents.
Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.
No. 191MP. . Roadside Improvement. 10 cents.
No. 272MP. . Construction of Private Driveways. 10 cents.
No. 279MP. . Bibliography on Highway Lighting. 5 cents.
Highway Accidents. 10 cents.
The Taxation of Motor Vehicles in 1932. 35 cents.
Guides to Traffic Safety. 10 cents.
An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.
Highway Bond Calculations. 10 cents.
Transition Curves for Highways. 60 cents.
Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.
No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.
No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.
House Document No. 272 . . . Toll Roads and Free Roads.
Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).
Report of a Survey of Transportation on the State Highways of Vermont (1927).
Report of a Survey of Transportation on the State Highways of New Hampshire (1927).
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).
Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).
Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.
Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.
Act III.—Uniform Motor Vehicle Civil Liability Act.
Act IV.—Uniform Motor Vehicle Safety Responsibility Act.
Act V.—Uniform Act Regulating Traffic on Highways.
Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MAY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossing Protected by Signals or Otherwise	
Alabama	$ 1,036,997	$ 1,033,332	14	2	1	$ 766,310	$ 766,227	6			$ 15,400	$ 15,400			4	$ 881,219
Arizona	317,016	316,972	1			201,045	198,341	3			2,930	2,930	1			397,816
Arkansas	182,388	182,199	3		3	824,603	820,955	4	1	4	381,594	381,152	6		7	607,550
California	1,343,969	1,339,199	10		2	1,371,661	1,192,630	6	1		25,524	25,524			11	1,701,560
Colorado	655,359	624,117	6		23	277,629	277,629	1		1	5,643	5,643			2	922,522
Connecticut	47,558	33,998			7	619,053	608,861	5	1		157,343	157,294	2			459,960
Delaware	7,839	7,839	2			20,794	20,794			4	186,356	153,408	1		12	490,500
Florida	428,094	428,094	2			196,653	192,154	2			35,832	35,832			10	1,426,798
Georgia	231,598	222,834	5		4	668,848	668,848	12	4	4	427,984	427,984	3	2	14	2,279,463
Idaho	204,578	172,785	3			345,006	314,413	4			53,156	53,156			22	452,353
Illinois	2,555,331	2,442,431	16	5	58	2,261,860	2,100,755	11	1	38	196,406	133,915			74	2,570,505
Indiana	869,394	861,372	3	1	97	806,789	806,789	5	2	42	97,172	97,172			38	1,295,880
Iowa	1,228,220	1,154,302	14		194	246,260	177,738	2		28	304,444	249,925	1		73	1,342,089
Kansas	1,000,135	992,198	12		11	795,782	795,304	11		3	185,919	185,919	1	2	13	1,272,748
Kentucky	611,305	600,627	8	4	12	995,759	995,759	9		2	233,426	233,426	5		13	634,375
Louisiana	493,976	493,958	4			460,321	406,826	5			627,885	570,153	11			789,393
Maine	348,233	345,696	3	2		209,733	209,733	2	1		139,758	139,758	1		5	279,164
Maryland	128,896	119,788	1		14	608,009	576,216	3	1		6,400	6,400			2	825,125
Massachusetts	491,527	490,263	4	2		31,039	31,039				320,890	310,839	1			2,025,042
Michigan	872,632	851,833	6	2	41	1,801,658	1,801,658	9	3	7	481,534	481,534		2	18	867,308
Minnesota	522,494	496,493	4	4	17	1,683,207	1,682,113	12	2		310,070	310,070	4	2	13	1,225,344
Mississippi	332,954	332,954	7			504,319	504,319	4	1		233,150	233,150	2			940,174
Missouri	450,998	449,119	3	1		1,367,377	1,367,377	8	3		1,319,348	863,928	3	2	1	1,521,518
Montana	842,767	842,206	9			290,905	206,333	4			178,767	178,767	3		4	425,734
Nebraska	734,287	732,099	19		40	946,195	946,195	8	2	9	38,117	38,117	1		4	748,388
Nevada	206,196	201,868		4	13	4,693	4,693				92,236	91,981	1		25	170,853
New Hampshire	102,433	101,921	7		1	181,248	181,212	4								392,776
New Jersey	463,987	463,987	1	2	1	542,654	542,654	3	1		225,737	225,737	2			1,529,653
New Mexico	123,381	122,378	3		1	230,046	230,046	2			20,902	20,902			6	645,923
New York	1,906,664	1,872,132	5	8		3,177,007	3,099,667	12	13		405,445	360,845		2	8	3,672,130
North Carolina	1,225,139	1,189,765	6	5	47	913,708	913,708	12	2	10	77,010	77,010		1	14	1,139,819
North Dakota	525,514	476,687	7	1		436,880	436,880	6		5	110,270	110,270	1		16	780,868
Ohio	545,780	530,780	8	1	4	2,155,153	2,084,201	12	2		701,970	686,780	2	1	13	3,010,809
Oklahoma	439,951	436,211	6		50	355,527	354,627	3	1		551,352	549,382	9		41	2,020,671
Oregon	173,428	171,720	2			307,391	218,374	3			37,918	32,000	1			558,332
Pennsylvania	1,676,971	1,465,072	2	3	2	1,344,225	1,344,225	12	1		948,724	701,000	6	1		4,843,667
Rhode Island	446,089	445,450	1	3		7,406	7,406				209,568	209,568		1		75,077
South Carolina	483,201	449,677	7	4	7	469,383	469,383	5	5		131,328	130,925		1	27	1,070,979
South Dakota	321,217	321,217	3	2	17	126,227	126,227	2	1		65,340	64,480	1		4	1,341,768
Tennessee	297,855	283,452		2	5	576,552	576,552	3	2		34,534	34,534	1			1,877,976
Texas	2,483,146	2,450,399	23	3	16	1,720,768	1,648,960	14		5	278,936	254,000	3		8	2,694,244
Utah	385,851	385,643	3		136	28,638	28,638			8	4,350	4,350			1	372,032
Vermont	32,093	27,207			7	206,402	206,402		2		12,043	12,043			3	252,060
Virginia	731,240	638,340	8	3	20	233,033	231,826	3	2		77,843	77,623	1		3	1,285,175
Washington	343,038	341,562	3	1	12	425,362	423,862	6	3		21,910	21,910			7	579,746
West Virginia	377,241	361,481	7		2	17,310	17,310	2			113,000	113,000	1		2	1,253,132
Wisconsin	889,268	878,511	9		11	1,004,757	958,634	8	3	2	274,499	267,661	1	1	9	1,388,345
Wyoming	139,774	120,314	1	1	8	89,337	89,337	1			327,458	294,000	3			350,475
District of Columbia	317,500	317,500	1			8,868	8,868				44,300	44,300	1			155,902
Hawaii	168,936	166,306	2	1	1	223,256	223,249	4			4,810	4,810			1	294,506
Puerto Rico	49,040	48,840	1			584,007	579,336	11								414,357
TOTALS	30,793,478	29,835,128	275	67	885	33,670,653	32,675,783	264	61	172	10,736,531	9,680,507	80	18	528	58,554,303

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Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 5 July 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

A STUDY OF MOTOR-VEHICLE DRIVERS AND SPEED IN CONNECTICUT[1]

By HARRY R. DeSILVA, Research Associate in Psychology, Driver Research Center, Institute of Human Relations, Yale University

FACTUAL DATA regarding the influence of speed on highway accidents are very scarce, although a great deal has been said and written about speed as a cause of accidents. Before this problem can be solved, it is necessary to find out who the speeding drivers are, how many there are, and when and where they drive. This can only be accomplished by coordinating the speeds of vehicles using the highways with certain facts regarding the individual drivers.

The first of a series of investigations to throw some light on speed as related to the characteristics of individual drivers, was conducted in Connecticut during the summer of 1939 by the Driver Research Center at Yale University, in cooperation with the American Association of Motor Vehicle Administrators, the United States Public Roads Administration, and the Connecticut Motor Vehicle, State Police, and Highway Departments. This study was the first serious attempt to gather facts about the characteristics of operators whose driving speeds were accurately measured. Being the first study of its kind, it is somewhat fragmentary; but in spite of its incompleteness it is offered as a pattern that can be altered or amplified in future studies.

The findings of this investigation, which involved a comparatively small number of drivers, must be considered as tentative until corroborated by studies in other States. A driver-speed investigation has recently been completed in South Carolina and similar studies are now under way in other States.

A survey was first made of various main roads in the vicinity of Hartford, Connecticut, to select tangent sections of highway relatively free from physical hazards that might influence vehicle speeds. A description of each of the selected locations is contained in table 1. All observations were made on weekdays between the hours of 10 a. m. and 4:30 p. m. The weather was clear and warm and the visibility excellent except during the studies at station 9 where there were intermittent showers.

Vehicle speeds were obtained with a speedmeter of the recording type described and illustrated in the April 1940 issue of PUBLIC ROADS. Efforts were made to conceal the truck housing the speedmeter so that it could not be seen by passing motorists. Two black rubber tubes, stretched across the road 24 feet apart, were used as detectors for the speedmeter. Very few drivers were aware of their presence either at a distance or when passing over them as they resembled expansion joints. Two men near the edge of the road but out of sight of oncoming cars read the registration numbers of all vehicles traveling in one direction and recorded them together with the vehicle speeds.

About a mile down the road and out of sight of the speedmeter location all vehicles traveling in this direction were stopped by a policeman who motioned them over to the side of the road where the drivers were questioned by an investigator. As about 2 minutes were required to question each driver, several men were used for this operation to prevent the formation of long lines during the hours of heaviest traffic. The questionnaire used is shown in figure 1.

At the end of the day the registration numbers on the questionnaires were matched with the registration numbers and speeds obtained at the speedmeter location. When there was any doubt about matching the registration numbers the questionnaire concerned was discarded. Two thousand six hundred and sixty-four questionnaires were identified and found to be complete enough for use in practically all of the subsequent analyses.

Upon completion of the field work, the information on the questionnaires was placed on punch cards and tabulated.[2]

OUT-OF-STATE DRIVERS TRAVELED FASTER THAN CONNECTICUT DRIVERS

The number of drivers studied at each location, their average speed, and number of miles traveled on the day interviewed are shown in table 2. The drivers of 74 light commercial vehicles were also interviewed but in this table, as in all other tables, they are excluded unless a separate classification is made for them. No data were collected for large trucks and busses.

TABLE 1.—*Description of study locations*

Station	Date 1939	Day of week	Location	Direction of traffic studied	Type of highway				Posted speed limit
					Surface type	Number of lanes	Width	Shoulders	
							Feet		*M. p. h.*
1	Aug. 8	Tuesday	U. S. Route 6A, 3 miles west of New Britain traffic circle.	West-bound	New concrete	2	20	5 feet oiled	(1)
2	Aug. 9	Wednesday	do	East-bound	do	2	20	do	(1)
3	Aug. 10	Thursday	do	West-bound	do	2	20	do	(1)
4	Aug. 15	Tuesday	State Route 10, 3 miles north of Cheshire	North-bound	Concrete	2	22	do	(1)
5	Aug. 16	Wednesday	State Route 10, 5 miles south of Cheshire	South-bound	Uneven asphalt	3	20	Extra wide	(1)
6	Aug. 17	Thursday	U. S. Route 6A, 3 miles west of New Britain traffic circle.	East-bound	New concrete	2	20	5 feet oiled	(1)
7	Aug. 23	Wednesday	State Route 15, 1 mile south of Vernon Center [2]	South-bound	Concrete	2	20	do	40
8	Aug. 24	Thursday	State Route 15, 1 mile south of Vernon Center	do	do	2	20	do	40
9	Aug. 25	Friday	State Route 2, 2 miles south of East Glastonbury.	do	Concrete (wet)	2	20	do	40

[1] Not posted; 50 miles per hour permissible.
[2] Down 3 percent grade.

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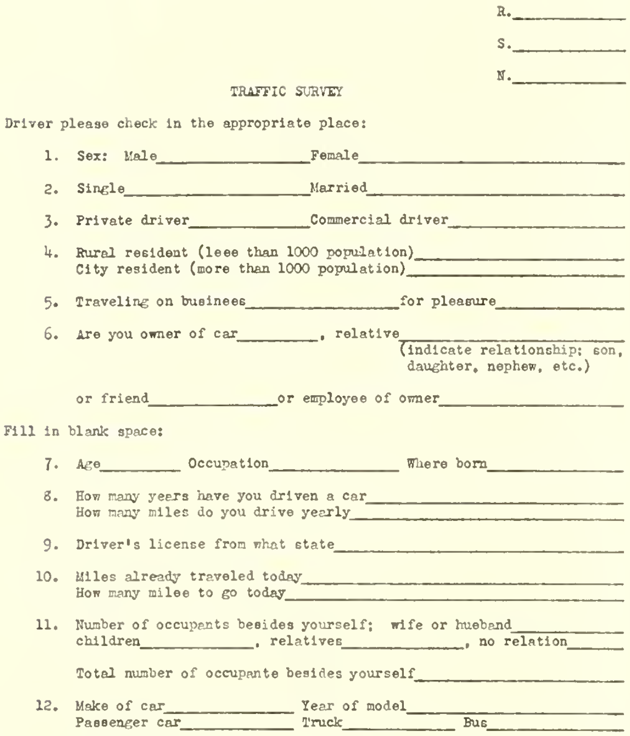
[1] Financed by a grant to Yale University from the Esso Safety Foundation.

[2] The Aetna Casualty and Surety Company generously helped in tabulating the data.

TABLE 2.—*Average speeds and total trip distances of vehicles observed at various study locations*

Station	Drivers interviewed				Average speed				Total average trip today			
	Men		Women		Men		Women		Men		Women	
	Connecticut	Out-of-State	Connecticut	Out-of-State	Connecticut	Out-of-State	Connecticut	Out-of-State	Connecticut	Out-of-State	Connecticut	Out-of-State
	Number	*Number*	*Number*	*Number*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*Miles*	*Miles*	*Miles*	*Miles*
1	103	66	28	9	41.3	43.6	39.8	42.9	92	240	58	265
2	101	63	31	16	43.9	46.8	44.2	44.6	96	231	74	199
3	133	52	32	11	41.2	43.9	39.3	40.8	99	231	87	219
4	176	71	29	7	41.2	43.1	39.8	42.3	118	193	80	217
5	151	31	31	6	39.7	40.5	39.9	40.4	88	167	75	162
6	243	149	47	30	44.7	46.3	43.3	43.9	116	240	92	263
7	95	204	24	48	44.5	47.2	41.1	47.5	160	291	113	271
8	104	290	23	74	46.2	46.2	41.2	45.2	140	280	135	296
9	73	23	12	4	47.3	48.4	49.2	40.6	118	160	94	193
Total	1,179	949	257	205	42.7	45.8	41.6	44.8	113	254	88	264
Combined	2,128		462		44.1		43.1		176		166	


R.____

S.____

N.____

TRAFFIC SURVEY

Driver please check in the appropriate place:

1. Sex: Male____ Female____
2. Single____ Married____
3. Private driver____ Commercial driver____
4. Rural resident (less than 1000 population)____
   City resident (more than 1000 population)____
5. Traveling on business____ for pleasure____
6. Are you owner of car____, relative____ (indicate relationship: son, daughter, nephew, etc.)

   or friend____ or employee of owner____

Fill in blank space:

7. Age____ Occupation____ Where born____
8. How many years have you driven a car____
   How many miles do you drive yearly____
9. Driver's license from what state____
10. Miles already traveled today____
    How many miles to go today____
11. Number of occupants besides yourself; wife or husband____
    children____, relatives____, no relation____

    Total number of occupants besides yourself____
12. Make of car____ Year of model____
    Passenger car____ Truck____ Bus____

FIGURE 1.—QUESTIONNAIRE FORM FILLED OUT IN SPEED STUDY.

Women drivers comprised 17.9 percent of both the Connecticut and out-of-State drivers interviewed. As women comprise 19.1 percent of all drivers registered in Connecticut, the sample of drivers obtained during the study was fairly typical in regard to the proportion of men to women drivers. Out-of-State drivers comprised 44.5 percent of the total sample. This large proportion of out-of-State drivers is not typical of traffic on all Connecticut highways and was no doubt caused by conducting the studies during hours when most Connecticut drivers are at work, during August when there is a relatively high percentage of tourist traffic, and on through routes carrying a relatively high percentage of out-of-State traffic. For this reason the data in most tables presented are generally segregated by Connecticut and out-of-State drivers.

At all stations, out-of-State men drove faster than Connecticut men and at all but station 9, where a small sample was obtained, out-of-State women drove faster than Connecticut women, the average difference being 3.1 miles per hour for men and 3.2 miles per hour for women.

Women drivers, both Connecticut and out-of-State, drove at practically the same average speed or at a lower speed than the men at all locations except station 9 where the women Connecticut drivers traveled 1.9 miles per hour faster than the men.

The average trip length for out-of-State drivers was much longer than that for Connecticut drivers. The average trip length was practically the same for out-of-State men and women but considerably shorter for Connecticut women than for Connecticut men.

The average speed for men drivers at the different stations ranged from 39.7 miles per hour at station 5, to 48.4 miles per hour at station 9. The high speeds at station 9 may be explained by the large number of drivers in a hurry to get to shore resorts Friday afternoon ahead of an oncoming storm. Under these conditions, the Connecticut women drove faster than the men.

The uneven road surface was probably a factor causing the low average speeds at station 5, but the low average trip distance and the low percentage of out-of-State drivers probably had a greater effect.

Knowledge of the speed distribution of vehicles is necessary for the proper design of highways. Likewise, information about drivers in various speed groups is of fundamental importance for proper licensing and regulation of drivers using the highways. Average values for a number of characteristics of drivers traveling in each 5-mile-per-hour speed group are shown in table 3.

A comparison between the speed distributions found in this study and in a 1933–34 speed study in Connecticut shows that the percentage of drivers in the speed groups between 40 and 50 miles per hour has increased. In the present study the percentage of drivers traveling less than 40 miles per hour was only half as large, and the percentage traveling more than 55 miles per hour about three quarters as large as in the 1933–34 study.

The highest percentage of both men and women drivers is found in the 40 through 44 mile-per-hour group, as shown in figure 2. The average age of both men and women drivers increases from the low-speed groups up to the 30-34 mile-per-hour group and begins to decrease as the 45-49 speed group is reached. The average age of drivers in the high- and low-speed groups is lower than in the middle speed group. The increase in age for the 70-mile-per-hour group may be attributed to the smallness of the sample. Women drivers as a group were 3 years younger than the men.

Table 3.—*Average characteristics of drivers in various speed groups*

Speed group, m. p. h.	Drivers included in study: Men		Women		Total		Drivers included in study in 1933–34 [1]	Driver's age: Men	Women	Driving experience: Men	Women
	No.	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*Pct.*	*Years*	*Years*	*Years*	*Years*
Below 19	2	0.1	0	0	2	0.1	0.2	36.0	------	15.0	------
20–24	11	.5	4	.9	15	.6	1.9	39.7	34.0	17.0	13.2
25–29	60	2.8	14	3.0	74	2.8	7.2	37.7	36.5	15.1	9.0
30–34	166	7.8	41	8.9	207	8.0	16.9	41.2	36.2	16.2	9.5
35–39	269	12.6	65	14.1	334	12.9	21.5	40.4	36.7	16.5	10.0
40–44	685	32.3	168	36.4	853	32.9	25.5	40.1	37.2	16.8	11.9
45–49	558	26.2	112	24.2	670	25.9	13.6	38.9	36.9	16.6	12.8
50–54	291	13.7	45	9.7	336	13.0	8.3	37.5	34.2	16.0	11.4
55–59	62	2.9	11	2.4	73	2.8	3.3	36.1	38.3	16.2	14.8
60–64	13	.6	2	.4	15	.6	1.3	29.2	34.0	11.5	12.5
65–69	4	.2	0	0	4	.1	.2	34.5	------	15.2	------
70 and over	7	.3	0	0	7	.3	.1	39.1	------	14.9	------
Total	2,128	100.0	462	100.0	2,590	100.0	100.0	39.3	36.6	16.4	11.6

Speed group, m. p. h.	Annual travel: Men	Women	Other occupants: Men	Women	Trip length today: Men	Women	Price classification of vehicles driven by men: Low	Medium	High	Other [2]	Age of vehicles driven by men
	Miles	*Miles*	*Number*	*Number*	*Miles*	*Miles*	*Number*	*Number*	*Number*	*Number*	*Years*
Below 19	15,000	------	2.0	------	46	-----	2	0	0	0	------
20–24	19,900	17,000	.5	1.8	84	96	7	3	1	0	4.2
25–29	17,400	8,800	1.1	1.1	107	74	42	11	5	2	
30–34	18,200	8,400	1.1	1.3	122	95	97	43	9	17	3.3
35–39	16,800	7,900	1.2	1.7	145	123	180	55	19	15	
40–44	19,500	10,100	1.2	1.7	167	157	440	163	51	31	2.4
45–49	18,800	10,600	1.2	1.5	202	230	342	154	46	16	
50–54	18,700	11,100	1.2	1.3	218	184	173	79	35	4	1.8
55–59	20,700	8,400	1.1	2.1	180	258	30	20	11	1	
60–64	28,700	12,000	1.2	0	253	77	8	3	2	0	1.2
65–69	11,500	------	1.8	------	194	------	3	1	0	0	
70 and over	20,700	------	1.3	-----	179	------	4	2	1	0	
Total	18,800	9,900	1.2	1.6	176	166	1,328	534	180	86	2.5

[1] Motor Vehicle Speeds on Connecticut Highway, by C. J. Tilden, D. L. Morris, T. M. C. Martin, and E. W. Russell. Yale University, Committee on Transportation, 1936. (Included 73,171 vehicles.)

[2] Not classified.

Even more interesting is the fact that women as a group averaged about 5 years less driving experience, and performed only about half the annual travel, of men. This fact bears out the common assumption that the average woman driver is not as experienced a driver as the average man.

Neither the annual travel nor the years of driving experience show a clear-cut trend for the different speed groups. The speed group into which drivers with high or low annual travel or driving experience fall seems to be a matter of chance. The average annual travel, especially for men, is exceedingly high when compared to the average annual travel of 8,870 miles obtained for vehicle owners in 17 States through the highway planning surveys, and the average of 12,090 miles for men drivers in Connecticut as obtained by a Connecticut Motor Vehicle Department questionnaire in 1939. The reason for the higher mileage is that in this study a sample of the more active drivers who use the road during the daytime of weekdays was obtained rather than a representative sample of all drivers. A sample of all drivers would include a larger percentage of nonowners who drive only when someone lends them a car and also a larger percentage of owners whose annual travel is relatively small.

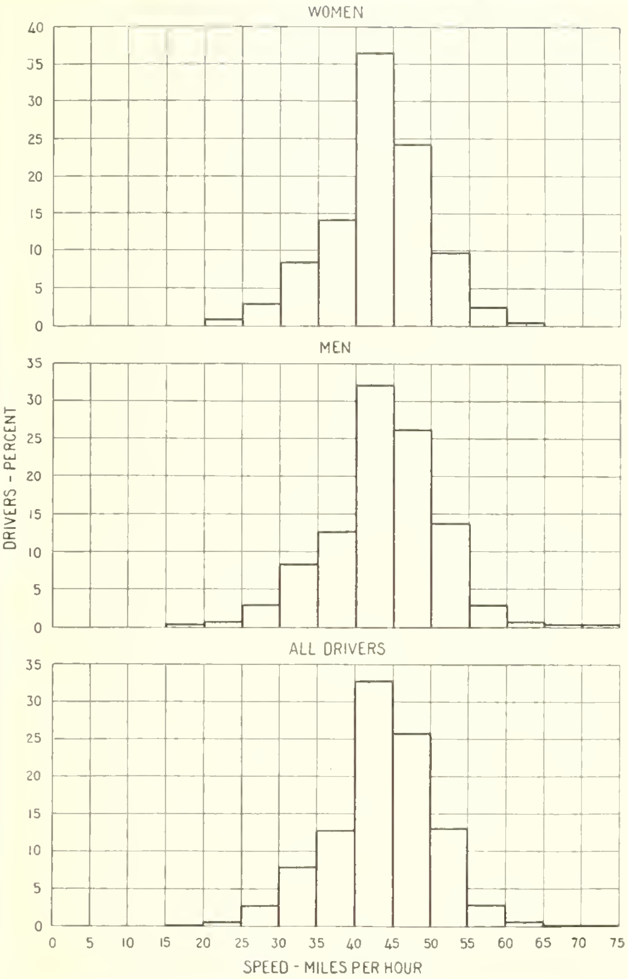


Figure 2.—Frequency Distribution of Speeds.

Cars driven by women had more occupants than those driven by men. An important factor in explaining this is that over half of the women were housewives, many of whom were accompanied by their children. The high- and low-speed groups of vehicles had about the same number of occupants as the average vehicle.

DRIVERS HAVING HIGH ANNUAL TRAVEL AND NEW CARS PREDOMINATE IN HIGH-SPEED GROUP

The "trip length today" shows a fairly definite tendency to increase as the speed increases, as shown in figure 3. In other words, the farther drivers intend to go, the faster they drive. This finding is amplified later in the report.

There is a very slight tendency for drivers of high-priced cars to travel faster than the drivers of low-priced vehicles. Low-priced vehicles as a group averaged 43.9 miles per hour, medium-priced vehicles, 44.7 miles per hour, and the high-priced vehicles 45.9 miles per hour. All three price groups are well represented in all speed groups but 11.2 percent of the low-priced vehicles, 10.7 percent of the medium-priced vehicles, and 8.3 percent of the high-priced vehicles were traveling less than 35 miles per hour. The percentages traveling over 50 miles per hour were 16.4, 19.7, and 27.2 for the low-,

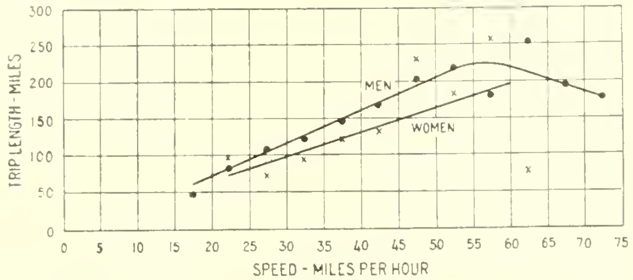


Figure 3.—Average Trip Length for Drivers Traveling at Various Speeds.

medium-, and high-priced groups, respectively. A distribution of the different classes of vehicle by sex and residence of driver indicates that women, especially Connecticut women, drive a proportionately greater share of the low-priced vehicles.

The average age of vehicle was found to vary inversely with vehicle speed (fig. 4). In other words, older cars predominate in the low-speed groups and newer cars predominate in the high-speed groups. The most obvious reason for this relationship is that the new cars will actually go faster, ride more smoothly and silently, handle better, and are generally in a better mechanical condition.

Table 4 shows characteristics of the individual drivers traveling faster than 60 miles per hour. Women comprised 17.8 percent of all drivers studied, but only 2 of the high-speed drivers (7.7 percent) were women.

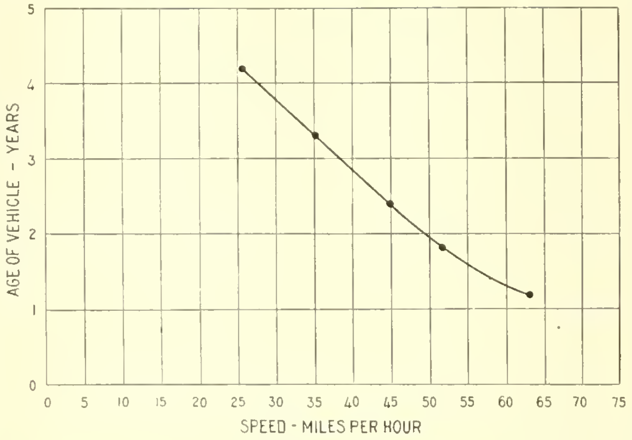


Figure 4.—Average Age of Vehicles Traveling at Various Speeds (Includes Only Vehicles Driven by Men).

The men traveling over 60 miles per hour included practically all groups into which the drivers could be classified. Married and single men, owners and nonowners, Connecticut and out-of-State drivers, and men on business and pleasure trips, as well as a variety of occupational groups are included. The majority were traveling considerable distances during the day, although a few were going short distances. Over one-third of the drivers traveling faster than 60 miles per hour were alone, and one-fourth of them were accompanied by only one passenger. Models of cars driven

Table 4.—*Characteristics of individual high-speed drivers*

Study station	Registration	Martial status	Residence	Purpose of trip	Relation to owner	Age of driver	Driving experience	Annual travel	Trip length today		Other occupants		Car driven	
									Traveled	Going	Number	Relation	Price	Year
MEN TRAVELING OVER 70 MILES PER HOUR														
						Years	*Years*	*Miles*	*Miles*	*Miles*				
8	Connecticut	Married	Urban	Business	Employee	39	19	25,000	200	212	0		High	1937
4	do	do	do	Pleasure	Owner	48	17	10,000	18	10	0		Medium	1939
6	do	do	Rural	Business	do	36	15	10,000	40	200	1		Low	1936
1	do	do	do	Pleasure	do	60	22	10,000	35	25	2	Wife and relative.	Medium	1939
3	Massachusetts	do	Urban	do	do	28	5	35,000	120	20	5	Relatives	Low	1939
3	do	Single	do	do	Friend	22	6	5,000	125	70	1	Friend	do	1939
8	New York	Married	do	Business	Owner	41	20	50,000	50	125	0		do	1939
MEN TRAVELING 65 TO 69 MILES PER HOUR														
9	Connecticut	Single	Urban	Pleasure	Son	18	2	12,000	15	75	0		Low	1939
4	do	do	do	do	Owner	31	12	12,000	50	80	1		do	1936
8	Massachusetts	Married	do	do	Employee	48	25	10,000	100	230	3		do	1935
2	do	do	Rural	do	Owner	41	22	12,000	127	100	3	Friends and relatives.	Medium	1939
MEN TRAVELING 60 TO 64 MILES PER HOUR														
6	Connecticut	Married	Urban	Business	Employee	27	11	40,000	70	103	0		Low	1939
6	do	do	do	Pleasure	Son	23	3	6,000	150	225	3	Relatives	do	1939
6	do	do	do	Business	Owner	46	29	30,000	40	60	1	Wife	High	1939
8	do	do	Rural	do	do	39	16	75,000	18	200	0		do	1939
9	do	Single	Urban	Pleasure	do	22	2	25,000	100	150	2		Low	1939
7	do	do	do	Business	do	24	10	40,000	200	150	0		Medium	1939
6	do	do	do	do	Son	18	2	5,000	15	40	2		Low	1939
8	Massachusetts	Married	do	do	Owner	26	10	20,000	95	230	0		do	1938
8	do	do	do	do	do	30	13	45,000	40	130	1		Medium	1939
7	do	do	do	Pleasure	Son	29	13	25,000	211	120	4	Wife and relatives.	Low	1939
7	do	Single	do	Social	Owner	26	8	12,000	100	150	2		do	1937
2	New York	do	do	Pleasure	do	37	20	25,000	100	200	0		Medium	1939
8	Pennsylvania	Married	do	do	do	32	12	25,000	272	120	1		Low	1938
WOMEN TRAVELING 60 TO 64 MILES PER HOUR														
5	Massachusetts	Single	Urban	Pleasure	Owner	38	15	12,000	25	90	0		Low	1936
4	Connecticut	Married	do	do	Daughter	32	10	12,000	20	20	0		Medium	1937

by the high-speed drivers ranged all the way from 1935 low-priced to new high-priced cars.

Outstanding characteristics of the high-speed drivers as a whole are that drivers with new cars, and drivers having an annual travel over 25,000 miles comprised far more than their proportionate share. This is clearly shown by table 5. Single men, nonowners, pleasure drivers, drivers on trips of over 100 miles, and drivers below 30 years of age also comprised a considerably larger portion of the high-speed drivers than of the total sample.

TABLE 5.—*Comparison of characteristics of men drivers in high-speed group with characteristics of all men drivers studied*

Classification	Percentage of high-speed men drivers	Percentage of total men drivers in study
Out-of-State	45. 8	44 6
Urban	83. 3	81. 5
Unmarried	33. 3	25. 0
Nonowners	33. 3	27. 4
Pleasure trip	54. 2	47. 5
Annual travel over 25,000 miles	50. 0	22 0
On trip over 100 miles	79. 2	67 5
Driving 1938 and 1939 cars	79. 2	31. 5
Driving high-priced cars	12. 5	8 5
Two or more passengers	37. 5	30. 5
Age group:		
16–19	8. 3	2 7
20–29	37. 5	29. 7
Driving experience, years:		
4 or less	16 7	9. 1
5–9	12. 5	12 0
10–19	45 8	37. 3

Table 6 shows data relating to the characteristics of drivers in each age group. In all age groups except those over 60, in which a very limited sample was obtained, men drove slightly faster than women. Commercial drivers, including only those driving light vehicles, traveled at speeds considerably lower than either the men or women drivers of noncommercial vehicles.

Men in the 16–19 year age group had a slightly higher average speed than drivers in the 20–29 year age group. Beyond the age of 40 there was a definite drop in average driving speed with increasing age, for men drivers. The younger single men traveled at slightly higher speeds than the younger married men. Beyond 40, married men traveled as fast as single men. The differences are, however, very small and figures on larger numbers must be obtained before much importance can be attached to this finding.

Interesting information is presented in table 6 for each age group regarding the driving speeds of owners and nonowners, and the percentage of drivers in each group that owned the cars they were driving. A separate breakdown of the 20–29 age group is shown for these factors in table 7.

TABLE 7.—*Characteristics of young drivers*

Age of driver, years	Drivers included in study		Speed						Owners	
			Men			Women				
	Men	Women	Owners	Nonowners	Total	Owners	Nonowners	Total	Men	Women
	No.	*No.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*Pct.*	*Pct.*
20	34	5	42. 3	43. 9	43. 6	51. 8	47. 3	48. 2	17. 6	20. 0
21	25	10	42. 6	43. 4	43. 1	46. 9	44. 9	46. 1	32. 0	60. 0
22	58	8	43. 1	47. 8	45. 5	------	45. 8	45. 8	50. 0	0
23	42	15	44. 3	45. 1	44. 6	43. 0	41. 0	41. 7	64. 2	33. 3
24	51	12	43. 3	47. 5	44. 7	41. 7	42. 3	42. 0	66. 6	58. 3
25–29	231	66	44 5	41. 8	44. 6	44. 3	41. 8	43. 0	68 8	46. 9

MEN HAD MORE DRIVING EXPERIENCE THAN WOMEN

As would be expected, the data show that for the sample obtained during this study very few of the men under 20 years of age were owners of the cars they were driving (see fig. 5). The percentage of owner drivers

TABLE 6.—*Characteristics of drivers by age groups*

Age of driver, years	Drivers included in study			Speed											Owners	
				Men					Women							
	Men	Women	Commercial	Single	Married	Owners	Non-owners	Total	Single	Married	Owners	Non-owners	Total	Commercial	Men	Women
	Number	*Number*	*Number*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*M.p.h.*	*Percent*	*Percent*
16–19	57	10	5	45. 1	--------	41. 0	45. 3	45. 1	39. 8	--------	58. 4	37. 7	39. 8	40. 1	10. 5	10. 0
20–29	441	116	28	44. 7	44. 1	44. 1	45. 3	44. 5	43. 1	44. 1	44. 3	42. 7	43. 4	40. 2	59. 6	43. 1
30–39	618	169	21	45. 4	44. 5	44. 7	44. 9	44. 7	43. 0	43. 1	42. 7	43. 6.	43. 1	39. 8	74. 9	55. 3
40–49	597	105	10	43. 3	44. 0	43. 7	44. 5	43. 9	43. 9	42. 6	43. 5	42. 4	43. 1	34. 7	79. 3	67. 3
50–59	269	50	6	43. 1	43. 1	43. 4	41. 9	43. 1	41. 7	43 2.	42. 3	43. 3	42. 6	37. 4	81. 4	81. 6
60–69	124	10	4	42. 8	42. 3	43. 0	40. 0	42. 4	40. 4	40. 6	42. 4	--------	42. 4	34. 6	81. 4	100. 0
Over 70	22	2	0			42. 1	40. 9	42. 0			44. 2	46. 4	45. 3	--------	90. 9	50. 0
Total	2, 128	462	74	44. 6	43. 9	43. 9	44 5	44. 1	42. 9	43. 1	42. 9	42. 8	42. 9	38. 8	72. 6	57. 7

Age of driver, years	Other occupants		Trip length today		Driving experience		Annual travel					Age of vehicles driven by men	Distribution of men drivers			
							Men						Studied			Registered in Connecticut
	Men	Women	Men	Women	Men	Women	Connecticut	Out-of-State	Total	Women	All men in Connecticut[1]		Connecticut	Out-of-State	Total	
	Number	*Number*	*Miles*	*Miles*	*Years*	*Years*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Years*	*Percent*	*Percent*	*Percent*	*Percent*
16–19	1. 6	1. 7	159. 4	118. 6	2. 0	1. 9	8. 300	10, 100	8, 900	6, 700	9, 900	3. 0	3. 1	2. 1	2. 7	4. 8
20–29	1. 2	1. 5	165. 0	162. 7	7. 6	6. 5	20, 100	16, 900	18, 852	8, 952	15, 300	3. 0	22. 8	18. 1	20. 7	29. 6
30–39	1. 1	1. 8	185. 0	160. 3	15. 4	12. 1	21, 500	19, 900	20, 680	10, 220	13. 100	2. 4	27. 5	31. 0	29. 0	26. 9
40–49	1. 2	1. 6	182. 0	176. 1	20. 5	14. 1	19, 200	19, 400	19, 234	10, 706	12. 000	2. 2	27. 0	29. 4	28. 1	21. 1
50–59	1. 1	1. 1	164. 9	178. 5	22. 8	15. 7	18, 500	17, 700	18, 081	8, 570	10, 400	2. 5	19. 6	19. 4	19. 5	17. 6
60–69	1. 2	1. 0	172. 5	149. 9	24. 9	21. 0	14, 100	13, 500	14, 452	10, 100	6, 800	2. 7				
Over 70	1. 2	1. 0	171. 4	275. 0	23. 9	28. 5			9, 700	6, 100		4. 2				
Total	1. 2	1. 6	175. 9	165. 8	16. 4	11. 6	19, 200	18, 200	18, 800	9, 900	[2] 12. 086	2. 5	100. 0	100. 0	100. 0	100. 0

[1] As obtained by Connecticut Motor Vehicle Department questionnaire in 1939.
[2] Average annual travel from highway planning surveys in 17 states was 8,870 miles in 1936.

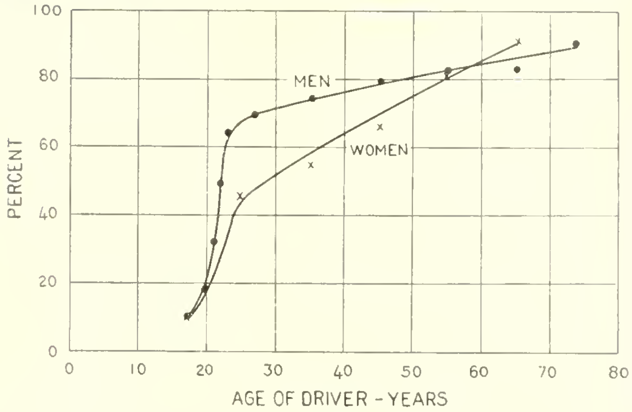


Figure 5.—Percentage of Drivers That Owned the Vehicles Being Driven.

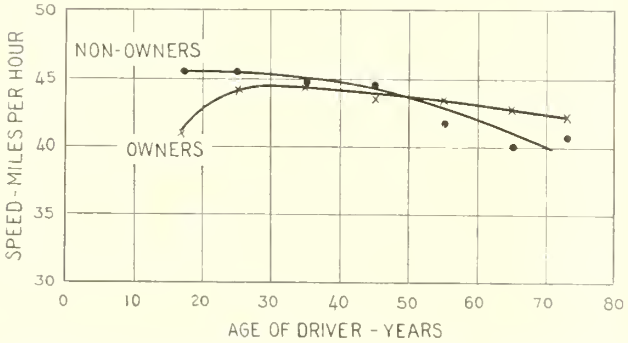


Figure 6.—Comparison of Average Speed with Age of Driver for Men Who Did and Men Who Did Not Own the Vehicles Being Driven.

increased rapidly from 17.6 percent at the age of 20, to 50 percent at the age of 22. Above 22 years of age, the percentage of owners increased gradually until at 70 nearly all men driving cars were owners.

The average speeds for men below 25 years of age who owned the cars they were driving were slightly lower than the average speeds for men of the same age who were nonowner drivers (see fig. 6).

The average annual travel for owners and nonowners was practically the same, but nonowners drove newer cars. The average age of the vehicles driven by nonowners was 2.32 years as compared to 2.61 years for vehicles owned by the drivers.

Table 8 indicates that the heaviest concentration of Connecticut nonowner drivers was in the 20–29 age group. Beyond the age of 30 the percentage of owner drivers is greater than nonowners. In the case of out-of-State drivers the percentage of owners was larger up to the age of 50, after which nonowners predominated. The proportion of owners among out-of-State operators reached 76.8 percent as compared to only 69.3 percent of owners among Connecticut motorists.

The number of women drivers studied was very small, so the results are probably influenced by chance to such an extent that the figures cannot be relied upon. However, the data shown by table 6 indicate that the small number of women in the 16–19 age group did not drive as fast as women in the other groups. There is practically no difference in the average speeds for owners, nonowners, single, or married women. Except for the exceedingly small group above 70 years old, the percentage of owners increased as the age increased.

There was no clear-cut relationship between the number of occupants and the age of the driver. Men under 20 years of age and women under 50 had the greatest average number of occupants, but whether this is typical can only be determined by further investigation.

The average trip length for young people was slightly lower than for other groups but there was no uniform trend.

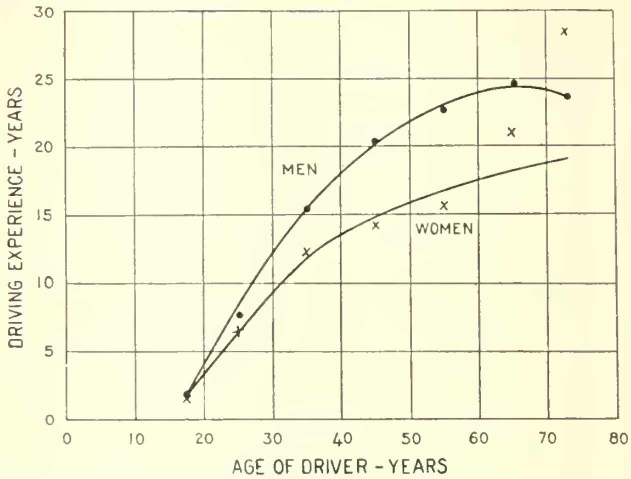


Figure 7.—Driving Experience for Drivers of Various Ages.

Table 8.—*Distribution of Connecticut and out-of-State men drivers by age and vehicle ownership*

Age, years	Connecticut		Out-of-State	
	Owners	Nonowners	Owners	Nonowners
	Percent	*Percent*	*Percent*	*Percent*
16–19	0.5	9.1	0.3	0
20–29	18.8	31.9	14.8	8.2
30–39	28.5	25.2	31.9	28.6
40–49	30.6	18.8	30.6	28.7
50–59	13.8	10.2	14.6	25.0
60–69	6.7	4.2	6.3	5.9
70 and over	1.1	.6	1.5	3.6
Total	100.0	100.0	100.0	100.0

The years of driving experience for men varied almost directly with the age of the driver up to about age 40 after which the years of experience ceased to advance materially with increased age (fig. 7). Young women up to the age of 25 had approximately the same length of driving experience as young men, but beyond 25 the average man had driven a car for more years than the average woman.

The annual travel for men less than 20 years old was much lower than for other age groups. Above 20, there was an increase up to the 30–39 age group and then a gradual decline with an increase in age. This was true for both Connecticut and out-of-State drivers. One reason why the 30–39 age group had the highest annual travel is probably the high percentage of salesmen drivers (30 percent) included in the study. Corresponding figures for all Connecticut men as obtained from the Connecticut Motor Vehicle Department questionnaire in 1939 do not indicate as great a variation for the different age groups and the average annual travel for all age groups was much lower. There was practically no relationship between the annual travel by women and their age although the young and old women drove fewer miles than women in the intermediate age groups.

The average age of the vehicles driven by men in the 40–50 age group was 2.2 years, which is lower than for any other age group. The average age of vehicle driven by the youngest drivers was 3 years, and 4.2 years for the drivers over 70. Although the average age of cars driven by elderly people was high, table 9 shows that they nevertheless drove a small percentage of the total number of old cars on the highway. Men in the 30–39 year group drove 27.8 percent of all cars over 10 years old. None of the drivers under 20 drove cars older than 10 years.

Table 9.—*Percentage of cars of various year models driven by men in various age groups*

Age of driver, years	Year model of car					
	1922–29	1930–31	1932–33	1934–35	1936–37	1938–39
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
16–19		3.5	4.9	3.6	3.5	2.5
20–29	13.9	32.9	36.6	25.0	22.2	18.5
30–39	27.8	25.9	23.2	26.2	26.2	30.3
40–49	19.4	21.2	19.5	22.6	26.9	32.0
50–59	19.5	8.3	12.2	17.3	13.7	10.3
60–69	11.1	8.2	3.6	4.1	6.5	5.7
70 and over	8.3			1.2	1.0	.7
Total	100.0	100.0	100.0	100.0	100.0	100.0

The data show that there were more middle-aged drivers among the active drivers on the road than among the Connecticut driving population as a whole. The younger drivers constituted a smaller percentage of the sample than they do of all licensed drivers in the State. Samples taken during the evening hours and over the week ends would undoubtedly show a larger percentage of young drivers on the highway. There was a larger percentage of younger persons among the Connecticut drivers than among the out-of-State drivers for both men and women, but the difference was slight.

In table 10, drivers are grouped by length of driving experience. For men, length of driving experience seemed to play no part in determining the speeds at which motor vehicles were driven. For women, the average driving speeds increased as the length of driving experience increased.

Table 10.—*Average speed and age of drivers classified by length of driving experience*

Driving experience, years	Drivers studied		Speed		Age	
	Men	Women	Men	Women	Men	Women
	Number	*Number*	*M. p. h.*	*M. p. h.*	*Years*	*Years*
1-4	193	68	43.7	42.0	24.1	29.1
5-9	256	110	44.1	42.7	27.7	32.0
10-14	371	124	44.4	42.4	34.4	36.3
15-19	423	79	44.1	43.8	39.7	40.7
20-24	495	58	44.1	44.3	45.2	43.1
25-29	271	20	43.7	45.9	49.4	
30-34	86		44.5		54.1	
35 and over	30		44.2		58.2	
Total	2,125	459	44.1	42.9	39.3	36.3

Although there were no women with more than 25 years of driving experience, a considerable number of men had been driving 25 years or more. Most women operators had been driving between 5 and 14 years, whereas the majority of men had been driving between 15 and 24 years. This fact cannot be accounted for by differences in the average ages of men and women drivers since the average age of women was less than 3 years below that of men.

The following tabulation, including only data for men, shows that the mileage driven increased with an increase in experience up to the 5–9 year group. The average driver with 5–9 years of experience drove practically the same annual mileage as the average of all drivers studied.

Driving experience, years:	*Annual travel, miles*
1	7,300
2	9,800
3	14,300
4	16,000
5–9	18,100
Average, all drivers	18,800

TRIP DISTANCE FOUND TO AFFECT DRIVING SPEED

In table 11, the operators are grouped according to the number of miles they had already traveled when they were interviewed, and in table 12 they are grouped according to the number of miles they still intended to drive that day.

Table 11.—*Number and average speed of drivers that had already traveled various distances on day of study*

Distance already traveled, miles	Drivers studied	Speed	Distance already traveled, miles	Drivers studied	Speed
	Percent	*M. p. h.*		*Percent*	*M. p. h.*
0–4	1.6	38.6	40–49	6.0	42.7
5–9	4.8	41.0	50–99	21.9	44.4
10–19	12.3	42.1	100 and up	36.2	45.6
20–29	9.3	42.1			
30–39	7.9	41.7	Total	100.0	43.9

Table 12.—*Number and average speed of drivers that had yet to travel various distances on day of study*

Distance yet to travel, miles	Drivers studied	Speed	Distance yet to travel, miles	Drivers studied	Speed
	Percent	*M. p. h.*		*Percent*	*M. p h*
0–4	1.0	39.9	40–49	5.7	42.1
5–9	3.6	40.0	50–99	20.8	42.7
10–19	7.7	41.6	100 and up	44.9	45.7
20–29	9.0	42.1			
30–39	7.3	42.8	Total	100.0	43.9

There was a definite relationship between average speed and trip distance. Drivers who had traveled short distances were driving at lower speeds than drivers who had traveled long distances. Similarly, drivers who still had long distances to go were traveling much faster than drivers who were near their destination. This could not be true had the total trip length been the same for all drivers.

A group of 698 operators who had already driven over 100 miles and were going over 100 miles more that day had an average speed of 46.3 miles per hour. Another group of 119 operators who had traveled less than 20 miles and had less than 20 miles still to go that day had an average speed of only 40.4 miles per hour. The five operators who had traveled less than 5 miles and had less than 5 miles still to go that day, had an average speed of only 36 miles per hour.

In table 13 operators are grouped according to their total reported daily mileages. The relationship between trip distance and speed holds quite well for all classifications (fig. 8). There were so few out-of-State drivers in the first groups that the average speed figures should be disregarded. A higher percentage of the Connecticut men than women were going over 100 miles. Nearly all out-of-State drivers, both men and women, were going over 100 miles.

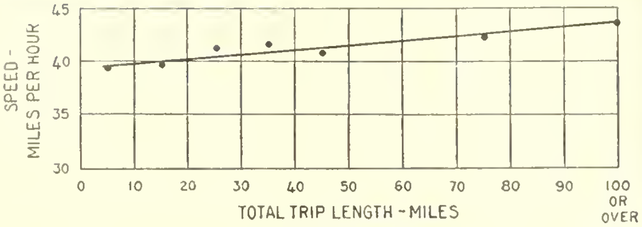


Figure 8.—Average Speed for Men Traveling Various Distances on Day of Study.

Table 13 also indicates that the sample of drivers is heavily overweighted in long distance groups, which explains the high annual mileage for the average driver. Since speed increases with an increase in trip length, the table also suggests that the average speed of the drivers in the sample was higher than the average speed would be for a more representative sample of all drivers in Connecticut.

Table 13.—*Number and average speed of drivers traveling various total distances on day of study*

Total distance today, miles	Number of drivers				Percentage of drivers				Speed			
	Connecticut		Out-of-State		Connecticut		Out-of-State		Connecticut drivers		Out-of-State Drivers	
	Men	Women	Men	Women	Men	Women	Men	Women	Men	Women	Men	Women
									M.p.h.	*M.p.h.*	*M.p.h.*	*M.p.h.*
0–9	5	3	---	----	0.4	1.2	-----	----	39.6	30.7	-----	-------
10–19	24	15	4	---	2.0	5.8	0.4	-----	39.7	41.2	45.6	------
20–29	47	23	4	---	4.0	8.9	.4	-----	41.4	39.8	43.0	----
30–39	76	30	---	2	6.4	11.7	----	1.0	41.6	39.2	---	41.1
40–49	101	20	3	4	8.6	7.8	.3	2.0	41.0	43.3	34.2	41.7
50–99	383	91	45	8	32.5	35.4	4.8	4.0	42.5	42.0	44.9	40.2
100 and up	543	75	892	188	46.1	29.2	94.1	93.0	43.7	42.9	45.1	45.2
Total	1,179	257	948	202	100.0	100.0	100.0	100.0	42.7	41.6	45.8	44.8

Table 14 shows the relative number and average speed of vehicles by year models. Out-of-State operators, most of whom were driving for pleasure, had newer model cars than Connecticut drivers, most of whom (especially the men) reported they are driving for business purposes (fig. 9). Thus, 82.7 percent of the out-of-State drivers had cars of 1936 model or later, as compared to only 69.7 percent of the Connecticut drivers. The Connecticut women included in the sample drove later model cars than the men, but out-of-State women drove about the same proportion of the newer cars as the out-of-State men.

A comparison between the percentage of registered vehicles in the year model group with the distribution of year models for the Connecticut drivers as obtained by the study indicates that the newer cars comprised a much larger proportion of the cars using the highway than they do of the total registration. Although 1938–39 models are only 14.5 percent of the registered vehicles, they comprised 33.9 percent of the sample. On the other extreme, vehicles of 1931 model or older comprised 27 percent of the registered vehicles but only 9.6 percent of the total sample.

Operators of late model cars, whether they were Connecticut or out-of-State drivers, traveled at higher speeds than motorists in older vehicles. It is particularly interesting to note that out-of-State operators drove both old and new cars at higher speeds than did Connecticut operators (fig. 10), indicating that the character of trip as well as the age of the vehicle had an effect on the speed.

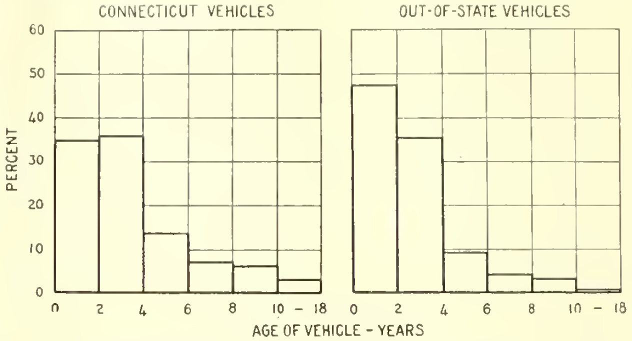


Figure 9.—Frequency Distribution of Vehicle Ages.

Table 14.—*Relative number and average speed of vehicles by year models*

Year model of vehicle	Vehicles driven by—						Vehicles registered in Connecticut	Speed	
	Connecticut drivers			Out-of-State drivers				Connecticut vehicles	Out-of-State vehicles
	Men	Women	Total	Men	Women	Total			
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*M.p.h.*	*M.p.h.*
1922–29	3.1	2.3	3.0	0.7	0.5	0.7	27.0	38.7	41.2
1930–31	7.2	3.9	6.6	3.5	2.0	3.2		39.3	40.7
1932–33	7.0	7.0	7.0	3.8	5.0	4.0	12.8	39.7	42.2
1934–35	14.2	10.9	13.7	9.3	9.9	9.4	17.4	42.0	45.6
1936–37	34.0	44.4	35.8	35.2	36.1	35.3	28.3	42.9	45.7
1938–39	34.5	31.5	33.9	47.5	46.5	47.4	14.5	44.0	46.3
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	44.1	42.9

Tables 15 and 16 show the number and average speed of vehicles driven by men according to the number of occupants and their relationship to the driver. Lone drivers traveled at higher speeds than drivers with passengers. However, the difference was more marked for Connecticut than for out-of-State drivers.

Table 15.—*Number and average speed of vehicles by the number of occupants*

[Includes only vehicles driven by men]

Occupants in addition to driver, number	Vehicles studied				Average speed of vehicles	
	Connecticut		Out-of-State		Connecticut	Out-of-State
	Number	*Percent*	*Number*	*Percent*	*M. p. h.*	*M. p. h.*
0	568	48.2	220	23.2	43.3	46.2
1	358	30.3	335	35.3	42.7	45.4
2	127	10.8	167	17.6	41.7	45.8
3	77	6.5	146	15.4	40.9	45.9
4	29	2.5	52	5.5	40.4	46.0
5	13	1.1	24	2.5	44.4	45.9
6	5	.4	5	.5	37.3	46.3
7	2	.2	0	0	41.1	---------
Total	1,179	100.0	949	100.0	42.7	45.8

There was a decrease in speed of Connecticut cars as the number of occupants increased to four (table 15). The data on out-of-State drivers show that the presence of passengers had little effect on speed. It is interesting to note that 1.7 percent of the Connecticut vehicles and 3 percent of the out-of-State vehicles had five or more passengers in addition to the driver.

Connecticut drivers traveling alone or with occupants that were of no relation traveled a little faster than those

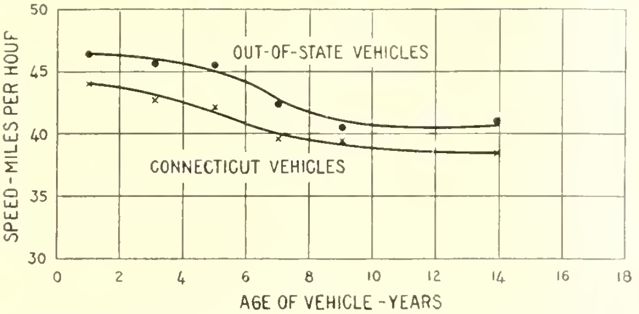


Figure 10.—Average Speeds of Vehicles of Various Ages.

Table 16.—*Number and average speed of vehicles by relationship of occupants to driver*

[Including only vehicles driven by men]

Relation of occupants to driver	Vehicles studied		Average speed of vehicles	
	Connecticut	Out-of-State	Connecticut	Out-of-State
	Number	*Number*	*M. p. h.*	*M. p. h.*
No other occupants	568	220	43.3	46.2
No relation	241	141	43.2	46.1
Children	19	17	42.3	42.8
Wife	119	194	41.5	44.9
Wife and children	72	169	40.2	45.7
Wife, children, and relatives	10	28	36.5	46.7

with members of their family (table 16). A much greater proportion of the out-of-State drivers than Connecticut drivers were accompanied by their wives. Thus, 68.8 percent of out-of-State men had their wives with them as compared to about one-third of the Connecticut drivers. The percentage of Connecticut drivers that had no passengers was more than double the corresponding figure for out-of-State drivers. The reason probably is that a majority of Connecticut drivers (60.3 percent) were on business trips while a majority of the out-of-State drivers (64.5 percent) were on pleasure trips. (See table 18.)

VEHICLES ENTERING CITY TRAVELED FASTER THAN VEHICLES LEAVING CITY

A number of characteristics for drivers in different occupational groups are shown in table 17. Chauffeurs were the fastest drivers and truck drivers in private cars were the slowest. The speed of salesmen was about the same as the speed of the average driver. Their annual travel was about one-third higher than reported by other drivers. The average annual travel for all drivers exclusive of salesmen was 15,900 miles, whereas the average annual travel for all operators, including salesmen, was 18,800 miles. The various occupational groups among women drivers had no marked differences in average speeds.

A separate occupational classification made of nonowner drivers under 30 years of age showed that a considerable proportion of the men were either students (23 percent) or salesmen (12 percent), and that the largest groups for women were teachers (28 percent) and housewives (20 percent).

Table 18 shows data for drivers grouped according to the purpose of the trip. Drivers on business trips traveled at approximately the same speed but shorter distances than drivers on pleasure trips. This holds for both men and women, and out-of-State as well as Connecticut drivers, although the difference in trip distance was proportionately less for the Connecticut than for the out-of-State drivers. A higher percentage of women than men were on pleasure trips. The difference was especially large for the Connecticut drivers.

Table 17.—*Average characteristics of passenger car drivers in various occupational groups*

MEN

Occupation	Drivers included in study		Speed traveled	Age of driver	Driving experience	Trip length	Annual travel	Other occupants
	Number	*Percent*	*M.p.h.*	*Years*	*Years*	*Miles*	*Miles*	*Number*
Chauffeurs	50	2.3	45.6	39.2	19.3	212	23,200	1.6
Professional	310	14.7	45.0	41.0	17.1	203	15,600	1.3
Students	69	3.2	45.0	20.3	3.5	195	8,500	1.6
Salesmen	641	30.1	44.3	40.1	18.1	171	25,400	.8
Unemployed	32	1.5	44.2	36.0	14.0	174	15,100	1.3
Truck drivers	30	1.4	41.8	30.3	12.2	188	41,200	1.4
Others	996	46.8	43.6	39.9	16.2	167	15,400	1.4
Total	2,128	100.0	44.1	39.3	16.4	176	18,800	1.2

WOMEN

Occupation	Drivers included in study		Speed traveled	Age of driver	Driving experience	Trip length	Annual travel	Other occupants
Professional	126	27.3	43.2	36.0	11.3	188	10,500	1.3
Housewives	216	46.8	42.9	39.4	12.8	155	9,100	1.8
Saleswomen	9	1.9	42.3	38.8	10.2	120	10,400	.4
Unemployed	5	1.1	42.3	24.0	6.0	191	12,200	1.4
Students	13	2.8	41.4	23.8	4.6	140	10,000	1.7
Others	93	20.1	43.6	33.4	10.5	167	10,700	1.3
Total	462	100.0	42.9	36.6	11.6	166	9,900	1.6

Table 18.—*Characteristics of drivers by purpose of trip*

Trip purpose	Drivers included in study				Percentage distribution of drivers			
	Connecticut		Out-of-State		Connecticut		Out-of-State	
	Men	Women	Men	Women	Men	Women	Men	Women
	Number	*Number*	*Number*	*Number*	*Percent*	*Percent*	*Percent*	*Percent*
Business	711	58	294	14	60.3	22.6	31.0	6.8
Pleasure	400	186	611	187	33.9	72.4	64.5	91.2
Combination	68	13	43	4	5.8	5.0	4.5	2.0
Total	1,179	257	948	205	100.0	100.0	100.0	100.0

Trip purpose	Speed				Trip distance			
	Connecticut		Out-of-State		Connecticut		Out-of-State	
	Men	Women	Men	Women	Men	Women	Men	Women
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*Miles*	*Miles*	*Miles*	*Miles*
Business	43.1	41.9	46.0	44.4	109	82	218	231
Pleasure	42.3	41.6	45.7	44.9	119	88	268	266

Table 19 summarizes the characteristics of drivers in a number of different group classifications. There were no significant differences between rural and urban operators in any of the factors studied. Eighty-two percent of both the men and women were urban residents.

Foreign-born men drivers had a slightly lower average speed than native-born men drivers. This may be due to the greater age or shorter trip distance for the foreign-born operators, who also reported a lower annual travel.

There were almost as many out-of-State drivers as there were Connecticut drivers. This indicates that the sample is not typical, since only on the main through roads would there be such a high percentage of out-of-State drivers.

The average speed of out-of-State men drivers was about 3 miles per hour faster than that of Connecticut drivers. This greater speed, however, is probably attributable to the much greater trip distance of the out-of-State drivers—254 miles as compared with 113 miles for the Connecticut drivers.

TABLE 19.—*Characteristics of drivers by different group classifications*

Classification of drivers	Drivers included in study		Average speed	Age of driver	Driving experience	Trip length	Annual travel	Other occupants
	Number	*Percent*	*M.p.h.*	*Years*	*Years*	*Miles*	*Miles*	*Number*
Rural residents:								
Men	394	15.2	43.9	38.8	16.4	173	18,200	1.1
Women	83	3.2	42.5	36.0	12.2	155	9,100	1.7
Urban residents:								
Men	1,734	66.9	44.1	39.4	16.5	177	18,900	1.2
Women	379	14.7	43.2	36.8	11.4	168	10,000	1.5
Foreign-born men	285	13.4	43.1	44.6	16.9	163	16,200	1.3
Native-born men	1,843	86.6	44.2	38.5	16.4	178	19,100	1.2
Connecticut:								
Men	1,179	45.5	42.7	38.9	16.1	113	19,200	.9
Women	257	10.0	41.6	36.0	10.7	88	9,300	1.4
Out-of-State:								
Men	949	36.6	45.8	39.7	16.8	254	18,200	1.5
Women	205	7.9	44.8	37.4	12.6	264	10,600	1.7

The difference in trip distance between Connecticut women and out-of-State women was even greater—264 miles for the out-of-State women as against 88 miles for Connecticut women. From these data it appears that the chief reason why out-of-State drivers traveled faster is that they were going farther.

In table 20 drivers are grouped according to the States in which their cars were registered. The average speed ranged from 46 miles per hour for Pennsylvania and New York drivers, to about 42 miles per hour for Maryland and Illinois drivers. Although Illinois and Michigan are much farther away from Connecticut than the other States represented, their drivers did not travel as fast as drivers from other States. Since the number of Michigan, Illinois, and Maryland drivers included in the sample was small, the particular character of the drivers involved may have much more to do with the average speeds than their place of residence.

TABLE 20.—*Average speed and trip distance for drivers from various States*

State	Drivers studied	Speed traveled	Trip length on day of study
	Number	*M. p. h.*	*Miles*
Pennsylvania	67	46.2	292
New York	276	46.1	248
Massachusetts	315	45.9	234
New Jersey	111	45.4	269
Maine	16	45.2	288
Connecticut	1,179	42.7	113
Michigan	11	42.6	271
Illinois	17	42.0	287
Maryland	12	41.6	320

A direct comparison was made between the speed of traffic traveling toward and away from Hartford, Conn., at a location a few miles from the city (at stations 1, 2, 3, and 6). At stations 1 and 3 the speed of traffic going away from the city (out-bound) was obtained, while at stations 2 and 6, the speed of traffic going toward the city (in-bound) was obtained. Table 21 shows that in-bound traffic consistently traveled from 2 to 4 miles per hour faster than out-bound traffic, regardless of whether the drivers were men or women, or from Connecticut or some other State. Speed studies made at the outskirts of Austin, Tex.,[1] and recent studies conducted by the Public Roads Administration in a number of States also show the same tendency for in-bound traffic to travel faster than out-bound traffic.

TABLE 21.—*Comparison of speeds for out-bound and in-bound traffic*

Direction of traffic	Drivers included in study				Average speed			
	Connecticut		Out-of-State		Connecticut drivers		Out-of-State drivers	
	Men	Women	Men	Women	Men	Women	Men	Women
	Number	*Number*	*Number*	*Number*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Out-bound	236	60	118	20	41.2	39.5	43.7	41.8
In-bound	344	78	212	46	44.5	43.7	46.4	44.1

The factor of trip distance does not account for the higher speed of in-bound drivers. According to table 22 the proportion of Connecticut drivers on long trips is slightly higher for the in-bound drivers than for the out-bound drivers, but there is no significant difference among the out-of-state drivers. Table 23 shows that very few of the in-bound drivers had Hartford for their destination since all sections of Hartford fell within the 0–9 mile range.

TABLE 22.—*Distribution of in-bound and out-bound drivers by total trip distance on day of study*

Total trip distance, miles	Out-bound drivers		In-bound drivers	
	Connecticut	Out-of-state	Connecticut	Out-of-state
	Percent	*Percent*	*Percent*	*Percent*
0–9	0	0	0	0
10–49	30.2	2.9	21.1	1.9
50–99	34.5	2.9	36.0	6.6
100–199	28.5	30.4	30.3	29.1
200–299	4.4	37.7	9.8	30.6
300 or more	2.4	26.1	2.8	31.8
Total	100.0	100.0	100.0	100.0

TABLE 23.—*Distribution of in-bound and out-bound drivers by remaining distance to go on day of study*

Remaining distance, miles	Out-bound drivers		In-bound drivers	
	Connecticut	Out-of-state	Connecticut	Out-of-state
	Percent	*Percent*	*Percent*	*Percent*
0–9	9.3	1.7	4.9	1.4
10–49	45.8	6.8	41.0	12.7
50–99	28.0	25.4	29.7	13.7
100 or more	16.9	66.1	24.4	72.2
Total	100.0	100.0	100.0	100.0

The simplest explanation of the differences in speed of in-bound and out-bound traffic seems to be that after traveling at higher speeds in the open country, drivers lose their sense of speed and, therefore, do not slow down when they approach a city until congested traffic actually impedes their progress. Drivers leaving a city en route and city drivers starting a trip are still "speed conscious" and increase their pace gradually until they are well away from the congestion of the city. It seems that having attained a high speed, drivers tend to continue at the same rate until they are compelled to slow down, whereas after being restrained by a city speed limit, they build up their speed very gradually, even after getting out of the city traffic.

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[1] Motor Vehicle Speeds on U. S. Highway 81 between Austin and San Marcos, Tex. The Information Exchange, May 15, 1939, No. 69, Texas Highway Department

Undoubtedly, the effect of proximity to a city on driving speed is modified by the size of the city and amount of traffic congestion in the city. Future investigations should attempt to throw more light on these points.

HIGH-SPEED DRIVERS HAD MORE ACCIDENTS AND TRAFFIC VIOLATIONS THAN MODERATE-SPEED DRIVERS

The accident and traffic violation records of 813 Connecticut drivers, 216 New York drivers, and 95 New Jersey drivers were obtained from the Connecticut, New York, and New Jersey motor vehicle departments. Since the names of the drivers were not obtained at the time the questionnaires were filled out, only the operators owning the cars they were driving could be identified from the registration numbers.

To study the relation between the operators' open road speed and their driving records, two groups were made. The high-speed group includes all operators traveling over the maximum permitted speed of 50 miles per hour on Connecticut highways. The moderate-speed group includes all operators who were traveling from 35 to 45 miles per hour.

Table 24 shows the number of operators involved in accidents, traffic violations, and speed violations, and the relative proportion of accidents and violations incurred by high-speed versus moderate-speed motorists. The reason that Connecticut operators had high accident records is that Connecticut accident records extend farther back than those of the other two States. In New York only fatal and personal injury accidents are legally reportable, while in Connecticut and New Jersey property damage accidents in excess of $25 must also be reported. This explains why New York residents had lower accident records than those of the other two States.

Table 24.—*Traffic accidents, traffic violations, and speed violations for high- and intermediate-speed drivers (includes only drivers who owned the cars they were operating)*

Speed group, miles per hour	Drivers included in study	Percentage of drivers studied having—			Traffic accidents per 100 drivers	Traffic violations per 100 drivers	Speed violations per 100 drivers
		Accident record	Traffic violation record	Speed violation record			
CONNECTICUT MEN							
	Number	*Percent*	*Percent*	*Percent*	*Number*	*Number*	*Number*
35–45	372	39.8	24.5	5.1	86.8	39.5	6.2
Over 50	82	56.1	40.2	20.7	137.8	86.6	31.7
NEW YORK MEN							
35–45	73	5.5	8.2	5.5	8.2	12.3	8.2
Over 50	38	13.2	15.8	7.9	15.8	15.8	7.9
NEW JERSEY MEN							
35–45	30	16.7	23.3	10.0	36.7	30.0	13.3
Over 50	18	33.3	27.8	16.7	72.2	44.4	16.7

The table reveals some interesting facts concerning the accident records of Connecticut drivers. A greater percentage of the drivers traveling at speeds above 50 miles per hour had been involved in accidents and, on an average, they had 58.7 percent more accidents per driver than drivers whose speeds were moderate (fig. 11).

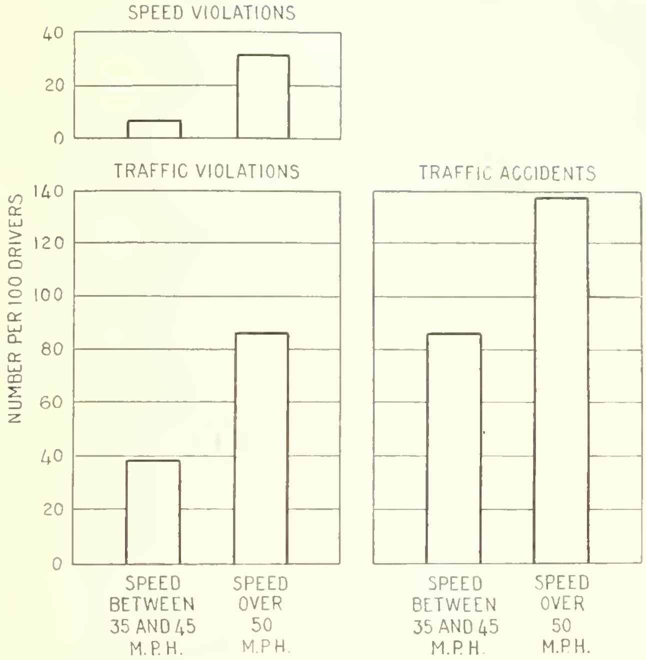

Figure 11.—Comparison of Number of Speed Violations, Traffic Violations, and Traffic Accidents for Moderate and High-Speed Connecticut Drivers Included in Study.

Similar results are shown for drivers with New York and New Jersey registrations. The number of records available were few but, with this reservation, it may be pointed out that the difference in accident records for fast and moderate-speed drivers was even greater than was the case with Connecticut drivers. The percentage of the New York drivers going over 50 miles per hour having accident records was almost 2½ times as great as for the moderate-speed drivers, while the percentage for the New Jersey high-speed drivers was twice as great as for the moderate-speed drivers. In both cases the accidents per driver were nearly twice as high for the high-speed as for the moderate-speed drivers.

The data in table 24 include speed violations as well as all other types of traffic violations. In Connecticut the high-speed group accounted for more than twice as many violations per driver as the moderate-speed group. Less marked differences appeared for New York and New Jersey operators.

The high-speed group in Connecticut had four times as high a percentage of speed violators as the moderate-speed group. On an average, each high-speed driver participated in five times as many speed violations as each driver in the moderate-speed group. Similar but less marked differences are noted for New York and New Jersey operators.

A comparison of the accident records of high- and moderate-speed Connecticut women drivers showed that 23 percent of the high-speed group had at least one traffic accident as against 20 percent for the moderate-speed group. The high-speed group averaged 46.2 accidents per hundred drivers as against 27.2 for the low-speed group. By comparing these figures with those for the men (table 24), it may be seen that both groups of women drivers averaged only one-third as many accidents per driver as the men. One reason for this large difference is that the average woman has not been exposed to traffic accidents as much as the average man, since her present annual travel is only about half as great and her total travel during the years that accident records were available was probably less than half the total travel of the average man.

A comparison of the average speeds of owners and nonowners of different ages (table 6) shows that young nonowners traveled faster than owners and that a great majority of young operators did not own the cars they were driving. These findings are especially significant in view of the fact that young people have a very high accident rate.

Not only did young people have more accidents than older people in proportion to the number driving but, according to the following tabulation, nonowners also had more than their share of accidents. Significant data pertaining to nonowner drivers are contained in the following tabulation:

	Percentage of drivers that were nonowners
Fatal accidents in Connecticut (1927–36)	51.3
All accidents in Connecticut (1927–36)	43.5
Drivers licensed in Connecticut (1927–36)	[1] 10.9
Drivers in speed study (1939)	31.2

[1] Estimated.

NONOWNERS, SINGLE MEN, AND DRIVERS OF NEW CARS PREDOMINATE HIGH-ACCIDENT GROUP

Nonowner drivers were involved in over half of the fatal accidents in Connecticut between 1927 and 1936 and in nearly half of all the accident reported. The proportion of owner and nonowner drivers in the Connecticut driving population has not been determined, so an estimate of the percentage of nonowners was based on the total number of licensed drivers and registered vehicles. Over the 10-year period, Connecticut had 89.1 percent as many registered cars as licensed drivers. If this figure represents the percentage of owners, nonowners had a much larger percentage of the accidents than they should have had in proportion to their number. The percentage of nonowners as obtained by this speed study is not so far from the figure for the percentage of accidents involving nonowners. Thus, there is an indication that nonowners have a higher accident rate than owners.

If these findings about vehicle ownership are verified for larger samples of the driving population, they will be useful in attempts to reduce the accident rate.

Of the Connecticut drivers included in this study there were 64 foreign-born men and 325 native-born men whose accident records were available. Fifty of the foreign-born men and 259 of the native-born men had been involved in accidents. The drivers with accidents constituted practically the same percentage of each group. However, the accident rate per mile driven was slightly higher for the foreign-born men, since their annual travel was only 84.8 percent of the annual travel for native-born men (table 19).

Practically the same percentage of single men as married men had been involved in traffic accidents. On the basis of mileage traveled, the accident rate was 10 percent lower for the married men.

Table 25 shows that the percentage of Connecticut registered vehicles involved in accidents during the first 6 months of 1939 was higher for new cars than for old cars. The new cars were involved in more accidents, were driven farther and at higher speeds, than were the older cars. Although many other factors are involved, the lower speed at which older cars are driven is probably a very significant factor in explaining the lower accident rate.

TABLE 25.—*Percentage of registered vehicles involved in accidents, average speed, and average annual travel for vehicles of different year models*

Model of vehicle	Connecticut registered vehicles involved in accidents [1]	Speed [2]	Approximate annual travel [3]
	Percent	*M. p. h.*	*Miles*
1929 and older	1.3	38.7	12,400
1930–31	1.3	39.3	14,600
1932–33	2.0	39.7	15,000
1934–35	2.5	42.0	17,500
1936–37	3.3	42.9	18,500
1938–39	3.2	44.0	22,600

[1] From Connecticut Motor Vehicle Department report Cars of Yesteryears. The figures on registered vehicles used in obtaining these percentages include transfers of registrations. Since a higher proportion of older automobiles are transferred than newer ones, the number of older model cars is higher than it should be. A careful check of the actual number of vehicles in use during a particular year will probably reveal that old-model vehicles show a higher accident rate than is indicated here.

[2] From table 14. Data for Connecticut men and women.

[3] Data only for Connecticut men who own cars.

There are a number of variables not covered by this study that may influence the speed of traffic. Since this investigation was conducted for the specific purpose of studying the effect on speed of factors relating to the driver, all other variables were eliminated, to as large an extent as possible, by the selection of locations and hours of study. Some of the other variables are:

1. Type, condition, and width of road surface.
2. Sight distance and highway alinement.
3. Type and density of traffic.
4. Diurnal, weekly, and seasonal changes.
5. Weather and temperature.
6. Speed regulations.
7. Regional differences.

Data necessary for a complete study of the effect of most of these variables are being obtained in connection with extensive highway-capacity and passing-practice studies being conducted by the Public Roads Administration. However, to obtain a complete picture of the characteristics of each group of drivers, it is essential that future speed studies of this nature include locations where driving conditions are not ideal. One particular group of drivers may travel at moderate speeds under ideal conditions but fail to exercise the same relative degree of caution under less favorable circumstances.

SUMMARY

The findings of this investigation are necessarily qualified by the conditions of this survey and must be considered as specifically pertinent only to drivers represented by the sample obtained. They are:

1. Out-of-state motorists drove newer cars, carried more passengers, and traveled faster than Connecticut drivers.
2. Young persons drove faster than older persons.
3. Nonowners, especially the younger persons, drove slightly faster than owners.
4. Women drove nearly as fast as men.
5. Drivers on long trips traveled faster than drivers on short trips.
6. Lone drivers and drivers with passengers to whom they were not related drove faster than drivers with passengers related to them.
7. Newer vehicles were driven faster than older vehicles.
8. Drivers who traveled faster than 50 miles per hour had been involved in more accidents, traffic

violations, and speeding violations than drivers who traveled between 35 and 45 miles per hour.

Other points which the study indicates for this sample of drivers are:

1. Men drove nearly twice as far each year, were not accompanied by as many passengers, and had been involved in more accidents than women.
2. Women drivers under 25 years of age had had as many years driving experience as men of the same age, but older women had had considerably less driving experience than men of the same age.
3. Newer vehicles were driven more miles each year and were involved in more accidents than older cars.
4. Drivers between 40 and 50 years of age owned a relatively large proportion of the new cars and had as many years of driving experience as men in the older age brackets.
5. Men between 30 and 50 years of age traveled more miles each year than either younger or older drivers.
6. The annual travel for the men with less than 2 years of driving experience was relatively low, but increased progressively up to 5 years of experience.
7. The majority of older drivers operated old cars.
8. Vehicles approaching a city traveled faster than vehicles that had recently left a city.

COMPILATION OF HIGHWAY RESEARCH ACTIVITIES AVAILABLE

The Highway Research Board has announced the publication of a compilation of the highway research activities of some 85 organizations, which include Federal Government agencies, State Highway Departments, colleges and universities, commercial laboratories, and trade associations. Entitled "Highway Research, 1920–1940," its preparation was a joint undertaking by the Committee on Research Activities of the American Association of State Highway Officials, F. V. Reagel, Chairman, and the Highway Research Board, R. W. Crum, Director.

The book contains sections on Highway Planning Survey, Economics and Finance, Design, Materials, Construction, Maintenance, Traffic, and Soils Investigations. Each section is further subdivided to classify the material in convenient form for reference. There are approximately 1,500 titles of research projects, together with the name of the agency reporting the investigation, a brief statement of the scope of the work, and its present status.

The book, paper-bound, is priced at $1.00 per copy and may be purchased from the Highway Research Board, 2101 Constitution Avenue, N. W., Washington, D. C.

205762

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JUNE 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 7,983,896	$ 3,863,858	307.5	$ 5,383,951	$ 2,676,140	174.2	$ 1,577,050	$ 785,270	57.8	$ 2,792,968
Arizona	2,856,253	1,991,768	150.7	1,125,844	744,693	47.7	341,609	155,595	15.2	1,684,118
Arkansas	5,275,011	4,153,122	236.8	2,393,379	1,213,226	115.9	701,625	345,671	34.0	1,475,127
California	6,178,220	3,266,446	107.0	8,018,617	4,158,986	122.8	3,248,310	1,711,300	59.7	2,899,895
Colorado	4,336,845	2,327,858	99.8	1,968,214	1,089,755	49.3	719,262	405,376	43.1	2,858,687
Connecticut	1,353,854	671,370	16.1	1,917,578	936,280	15.8	758,738	369,977	6.1	1,199,263
Delaware	951,438	455,106	31.5	1,277,655	638,504	11.3	900,394	423,082	21.3	1,048,125
Florida	3,640,769	1,817,537	41.7	3,707,680	1,843,561	114.7	786,872	393,436	11.4	2,464,359
Georgia	5,470,047	2,620,915	277.2	5,509,149	2,754,574	286.0	5,447,520	2,724,260	190.7	5,113,040
Idaho	2,436,176	1,429,520	147.9	1,066,142	652,672	67.1	564,127	294,707	83.6	1,955,063
Illinois	10,511,990	5,151,537	228.5	8,347,898	4,173,784	176.0	1,966,300	982,420	49.2	4,063,844
Indiana	4,756,066	2,290,863	93.7	7,220,581	3,603,884	150.7	1,720,904	860,400	23.8	2,332,955
Iowa	4,871,246	2,294,632	229.4	4,572,482	2,098,627	143.8	3,070,418	1,443,950	106.8	1,241,489
Kansas	4,446,514	2,152,020	244.2	4,529,183	2,280,577	257.6	4,684,483	2,325,736	314.8	4,114,901
Kentucky	4,057,997	2,005,897	130.6	3,863,166	1,930,027	76.6	1,452,267	726,133	66.1	3,142,407
Louisiana	1,748,247	870,857	54.0	11,834,155	2,960,476	37.3	2,576,521	1,276,640	60.0	2,921,124
Maine	2,300,239	1,124,922	57.3	971,947	485,973	19.8	935,236	467,618	24.8	680,386
Maryland	2,946,809	1,410,105	39.5	2,468,276	1,155,820	42.2	1,969,258	971,529	11.6	1,325,837
Massachusetts	3,152,338	1,573,441	25.0	2,017,840	1,005,372	18.6	2,425,928	1,207,014	17.4	2,822,159
Michigan	5,365,187	2,557,444	128.0	7,436,205	3,635,101	220.9	3,362,060	1,681,030	105.0	1,965,483
Minnesota	5,947,894	2,914,353	389.7	5,137,334	2,550,825	333.2	3,150,074	1,571,754	221.8	4,165,279
Mississippi	6,548,179	2,332,989	274.5	6,158,478	2,683,845	294.5	1,443,960	633,680	77.1	2,532,032
Missouri	4,573,116	2,259,656	181.0	5,509,698	2,717,661	203.6	4,631,506	1,810,176	108.0	4,987,762
Montana	3,876,387	2,188,757	237.0	3,838,952	2,173,522	254.8	1,024,134	578,103	54.5	4,087,726
Nebraska	5,418,390	2,587,274	468.7	5,542,441	2,667,672	614.9	2,023,386	1,011,693	276.7	3,127,504
Nevada	1,200,855	1,009,489	57.8	1,524,039	1,312,373	74.5	758,680	660,977	29.8	1,013,728
New Hampshire	1,036,250	508,948	29.7	1,317,360	648,274	34.5	543,604	206,060	10.8	925,691
New Jersey	1,754,260	865,896	15.5	5,505,948	2,752,744	41.8	172,620	86,310		2,145,319
New Mexico	2,631,111	1,600,580	211.4	1,805,829	1,114,077	107.0	544,640	328,309	35.3	1,643,995
New York	9,981,172	4,791,056	187.1	14,523,741	7,047,485	224.5	3,303,207	1,422,109	46.6	3,065,759
North Carolina	6,150,140	3,059,170	361.5	5,609,971	2,804,617	267.5	710,120	355,060	41.4	2,566,686
North Dakota	391,472	207,911	50.9	2,841,088	1,602,169	216.9	3,965,704	2,043,326	353.3	3,247,421
Ohio	7,549,042	3,715,068	88.7	10,946,455	5,449,793	107.3	5,299,630	2,648,915	44.4	5,301,074
Oklahoma	3,932,851	2,082,955	167.0	2,912,740	1,543,221	74.6	2,678,740	1,365,321	133.8	4,160,373
Oregon	3,449,128	2,065,015	123.8	3,627,295	2,176,497	152.8	512,491	305,363	12.0	1,482,545
Pennsylvania	10,756,517	5,145,525	116.8	10,320,863	5,113,518	122.6	4,650,292	2,300,876	32.4	3,476,189
Rhode Island	692,884	335,467	8.1	1,219,366	608,715	11.5	529,550	264,335	6.0	996,396
South Carolina	2,768,900	1,242,108	86.6	2,287,133	1,099,668	131.0	1,043,603	489,440	127.7	2,469,592
South Dakota	3,560,226	1,987,860	352.4	3,736,790	2,189,060	476.9	2,065,850	1,162,260	338.0	3,291,641
Tennessee	5,317,443	2,579,070	116.3	3,241,914	1,620,957	83.8	1,278,090	639,045	51.6	4,299,748
Texas	16,464,303	8,092,087	866.2	8,858,213	4,381,144	469.9	1,584,930	780,460	80.6	8,218,579
Utah	2,678,494	1,867,809	128.3	824,265	603,248	56.2	679,450	471,769	30.6	1,129,748
Vermont	741,568	350,426	18.4	1,331,658	665,807	40.9	517,087	257,547	10.9	356,275
Virginia	2,942,471	1,452,451	89.3	2,921,061	1,401,182	70.0	708,341	322,207	22.7	2,315,395
Washington	2,745,647	1,416,480	41.8	4,044,338	2,132,929	75.8	658,965	342,600	18.3	1,198,580
West Virginia	2,443,858	1,269,384	58.3	2,175,000	1,080,285	54.6	2,103,290	1,051,391	74.7	1,876,950
Wisconsin	5,150,897	2,525,683	187.6	5,215,376	2,560,898	161.9	1,569,103	749,535	84.7	3,644,541
Wyoming	1,934,933	1,189,596	200.2	1,579,651	1,004,436	147.8	395,537	252,822	67.4	1,185,041
District of Columbia	708,480	351,590	5.5	211,524	105,762	2.2	435,300	181,538	3.3	409,235
Hawaii	1,099,288	504,401	15.9	238,350	121,912	2.8	686,141	341,216	11.6	1,486,733
Puerto Rico	778,288	364,650	16.7	1,389,245	686,970	25.6	245,114	121,830	.6	761,970
TOTALS	209,863,586	106,892,922	7,799.1	212,026,058	104,659,298	7.084.8	89,122,021	44,307,171	3,709.0	129,670,737

U. S. GOVERNMENT PRINTING OFFICE

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JUNE 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,092,869	$ 436,439	48.3	$ 890,192	$ 443,270	46.4	$ 239,920	$ 119,950	11.3	$ 538,292
Arizona	417,901	298,394	51.8	105,397	55,108	10.5	139,904	101,010	.8	299,840
Arkansas	1,017,413	854,162	98.3	107,875	64,061	11.2	223,123	111,356	22.4	242,552
California	956,655	515,699	44.8	651,515	354,581	24.9	411,897	222,172	17.1	839,479
Colorado	1,132,362	568,158	37.9	54,923	8,092	.6	159,597	89,949	1.3	192,177
Connecticut	172,310	72,417	2.9	367,226	178,009	4.6				206,069
Delaware	84,115	39,067	17.5	69,537	34,768	7.8	98,995	41,350	7.7	268,125
Florida	904,133	443,690	31.3	29,916	14,958	.6	365,957	182,979	6.8	425,456
Georgia	363,706	172,132	44.2	468,089	234,045	57.3	498,287	234,143	28.2	1,131,926
Idaho	581,376	322,698	57.9	4,503	2,752		84,690	51,169	16.6	244,207
Illinois	1,667,905	747,611	103.2	1,758,985	862,318	73.3	513,400	256,067	13.2	408,283
Indiana	896,572	423,398	77.7	409,170	203,381	29.1	161,406	80,600	8.1	945,980
Iowa	1,048,264	500,412	222.8	1,406,733	669,575	255.4	937,574	442,760	196.2	447,293
Kansas	278,389	139,178	48.6	498,460	252,139	7.5	551,712	275,856	64.7	1,303,039
Kentucky	1,384,299	401,514	95.6	533,852	200,130	41.4	631,506	180,700	46.5	339,278
Louisiana	923,473	445,833	79.6	297,929	148,910	25.7				453,400
Maine	470,613	223,934	26.5	116,974	57,698	6.4	168,450	79,067	9.3	31,214
Maryland	306,943	144,266	20.2	131,996	65,998	5.5	62,000	31,000	4.9	417,585
Massachusetts	373,212	185,203	9.2	540,478	267,607	13.1	112,640	56,320	.3	486,733
Michigan	1,510,551	734,265	121.6	1,203,309	605,044	88.8	442,030	221,015	40.6	630,105
Minnesota	959,349	465,021	122.3	422,540	178,683	48.8	518,184	259,092	76.8	1,160,248
Mississippi	766,500	376,934	70.7	657,362	323,181	22.6	304,900	140,915	18.2	573,008
Missouri	1,124,147	554,417	169.8	548,630	274,315	55.4	291,700	118,619	53.2	767,826
Montana	1,024,043	574,247	95.5	445,129	251,761	54.1	280,366	158,210	28.7	641,509
Nebraska	1,131,239	529,803	222.9	527,521	263,601	76.2	458,592	229,296	63.6	297,846
Nevada	247,307	205,751	31.5	136,544	117,694	28.8	18,110	15,788	4.8	193,328
New Hampshire	113,324	55,299	4.3	85,641	40,146	2.2				197,644
New Jersey	355,458	174,500	12.2	559,540	279,770	18.3	31,230	15,495	1.3	524,785
New Mexico	469,280	287,227	42.1	367,851	229,448	26.9	150,797	72,336	.9	190,111
New York	1,966,117	946,407	95.0	2,169,035	1,049,440	69.0	691,100	263,450	14.2	395,619
North Carolina	1,053,159	520,851	109.5	981,563	491,748	85.9	139,080	57,804	12.5	328,477
North Dakota	114,601	59,497	8.2	62,121	34,895		149,983	80,390	3.8	1,014,688
Ohio	703,493	347,369	41.8	2,435,916	1,224,668	89.9	835,340	395,290	18.3	865,205
Oklahoma	531,016	276,162	36.1	581,706	308,724	43.8	361,180	166,526	16.5	924,501
Oregon	761,139	435,942	81.6	263,618	140,424	36.2	179,828	104,226	26.7	302,774
Pennsylvania	2,339,648	1,125,227	128.4	1,871,190	929,490	57.9	459,523	228,999	12.0	234,175
Rhode Island	93,827	46,890	2.2	230,054	115,001	3.6				95,049
South Carolina	589,454	235,287	56.9	651,540	250,828	61.0	177,519	74,850	56.4	227,521
South Dakota	27,371	15,025	4.1	3,624	3,624					1,280,913
Tennessee	956,289	420,211	32.3	114,396	57,198	7.9	29,896	14,948	3.0	1,006,149
Texas	2,896,756	1,425,706	328.9	1,151,693	562,938	153.3	338,958	149,175	42.2	1,122,102
Utah	340,753	200,505	46.7	6,700	5,000		187,609	98,600	19.3	193,707
Vermont	160,963	75,034	6.5	390,132	126,430	14.1	80,759	13,125	3.5	35,000
Virginia	711,983	341,341	68.3	551,564	245,012	32.8	31,750	10,000	.6	362,797
Washington	597,346	310,356	51.0	387,982	203,939	24.8	91,956	49,100	3.4	340,572
West Virginia	234,628	115,713	13.1	283,919	141,434	15.4	102,508	51,254	5.7	446,849
Wisconsin	997,327	495,708	36.7	525,424	262,350	7.8	340,406	167,870	20.4	686,667
Wyoming	470,702	286,620	26.0	389,051	232,715	39.0	55,850	35,699	3.9	150,806
District of Columbia	122,504	60,900	1.5	3,192	1,096		48,200	24,100	.6	60,154
Hawaii	193,032	96,211	6.2	278,748	139,878	8.7				157,036
Puerto Rico	30,220	14,440	2.2	302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	37,666,036	18,743,071	3,194.4	27,033,210	13,355,515	1,808.5	12,213,600	5,799,760	1,008.6	24,708,507

U. S. GOVERNMENT PRINTING OFFICE [illegible]

U. S. GOVERNMENT PRINTING OFFICE 1940

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JUNE 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			
			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 1,051,467	$ 1,047,240	14	2	1	$ 766,310	$ 766,227	6			$ 18,400	$ 18,400			5	$ 864,506
Arizona	317,016	316,972	1			201,045	198,841	3			2,930	2,930	1			397,816
Arkansas	184,930	184,741	3		4	1,004,709	1,000,672	6	1	4	214,569	214,516	4		9	589,253
California	1,679,436	1,670,290	11		3	1,057,831	879,350	5	1		22,524	22,524			10	1,687,055
Colorado	658,131	626,888	6		25	277,100	277,100	1			3,401	3,401			1	922,522
Connecticut	47,558	33,998			7	619,053	608,861	5	1		157,343	157,294	2			459,960
Delaware	7,839	7,839	2			59,822	59,822			11	125,767	125,767	1		6	479,112
Florida	428,094	428,094	2			196,653	192,154	2			134,380	134,050	2		11	1,328,581
Georgia	355,730	346,967	8	1	7	567,440	567,440	9	4	4	729,419	729,419	6	2	13	1,955,303
Idaho	312,690	280,535	4			204,098	200,668	5			74,256	74,256			29	439,501
Illinois	2,610,426	2,471,622	16	5	76	2,199,248	2,055,105	11	1	23	679,912	515,170		1	80	2,205,709
Indiana	871,744	855,554	3	1	103	814,808	814,808	5	2	44	85,049	85,049			33	1,305,684
Iowa	1,242,286	1,170,981	14		197	315,885	262,288	3		30	328,636	289,659	3		64	1,201,126
Kansas	1,001,808	993,871	12		13	874,993	874,515	12		2	105,445	105,445		2	12	1,273,339
Kentucky	696,370	686,291	9	4	16	1,061,316	1,061,316	11		1	110,453	110,453	2		10	606,126
Louisiana	513,679	513,661	6			440,619	387,123	3			627,884	570,153	11			789,393
Maine	415,253	412,716	4	2		143,637	143,637	1	1	1	163,120	163,120	1	1	5	254,878
Maryland	128,896	119,788	1		14	608,009	576,216	3	1		32,900	32,900		1	2	798,625
Massachusetts	491,527	490,263	4	2		352,301	341,878	1								2,025,042
Michigan	885,083	864,284	6	2	45	1,723,506	1,723,506	9	3	7	463,134	463,134		2	17	951,409
Minnesota	521,669	495,669	4	4	17	1,836,120	1,835,026	12	3		336,218	336,218	4	3	13	1,047,107
Mississippi	584,354	584,354	8			436,219	436,219	4	1		76,200	76,200	3		1	913,824
Missouri	697,868	695,989	5	1		1,120,507	1,120,507	6	3		1,319,348	863,928	3	2	1	1,521,518
Montana	851,419	843,820	9			266,174	196,534	2			178,767	178,767	3		4	433,920
Nebraska	970,081	966,187	22		43	730,583	730,583	6	2	6	18,258	18,258			4	749,771
Nevada	204,930	200,602	1	3	13	46,041	45,799	1			47,978	47,978			25	175,016
New Hampshire	102,433	101,921	7		1	181,248	181,212	4								392,776
New Jersey	707,902	707,902	2	3	1	298,739	298,739	2			515,107	515,107	3			1,240,283
New Mexico	123,381	122,378	3		1	230,045	230,045	2			20,902	20,902			6	645,923
New York	1,906,664	1,872,132	5	8		3,517,649	3,456,247	12	15		399,165	334,965		2	8	3,341,430
North Carolina	1,303,572	1,267,940	7	5	47	861,683	861,683	11	3	12	243,740	243,740	2		12	947,316
North Dakota	529,914	481,087	7	1	2	486,560	486,560	7		4	56,190	56,190			15	780,868
Ohio	545,780	530,780	8	1	4	2,772,313	2,701,361	14	3	1	54,390	54,390			12	3,026,039
Oklahoma	439,951	436,211	6		50	466,327	465,427	4	1		527,498	522,112	9		45	1,937,141
Oregon	304,186	301,184	3			219,579	125,939	3								553,304
Pennsylvania	1,676,971	1,465,072	2	3	2	1,475,807	1,466,331	14	1		685,751	685,751	5	1		4,736,810
Rhode Island	447,437	446,798	1	3		194,789	194,789		1							95,913
South Carolina	566,584	533,060	7	7	11	427,918	427,918	5	2	6	89,410	89,007		1	17	1,070,979
South Dakota	328,512	312,772	3	2	17	164,272	163,412	3	1		118,450	102,500	2		4	1,275,007
Tennessee	664,545	650,142	2	2	5	209,862	209,862	1	2		159,275	159,275	1	1		1,753,235
Texas	2,728,231	2,690,719	25	3	19	1,477,131	1,405,323	12		2	796,366	704,530	8	1	8	2,249,732
Utah	394,979	394,694	3		140	50,171	50,171			13						345,798
Vermont	32,093	27,207			7	206,402	206,402		2		15,472	15,472			5	248,630
Virginia	819,836	726,883	9	3	20	212,501	211,055	3	2	3	97,068	96,848	1		5	1,198,319
Washington	418,877	417,401	5	2	12	353,819	352,319	4	2	1	17,614	17,614			6	579,746
West Virginia	390,641	374,881	9		2	3,910	3,910				113,000	113,000	1		2	1,253,132
Wisconsin	889,269	878,511	9		11	1,269,895	1,225,773	9	4	9	15,110	15,110			4	1,373,756
Wyoming	139,774	120,314	1	1	8	377,238	377,238	4			179,518	177,183	1			179,391
District of Columbia	317,500	317,500	1			8,868	8,868				48,000	48,000	1			152,203
Hawaii	198,156	195,526	4	1	1	194,036	194,029	2			9,494	9,494			2	289,821
Puerto Rico	49,040	48,840	1			584,007	579,336	11								414,857
TOTALS	33,757,112	32,731,071	305	72	945	34,168,796	33,240,144	259	63	184	10,217,811	9,320,179	80	20	506	55,457,505

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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D. M. BEACH, *Editor*

Volume 21, No. 6 August 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

MARGINAL LAND ACQUISITION FOR HIGHWAYS

A DISCUSSION OF THE PROBLEMS INVOLVED IN THE ACQUISITION OF EXCESS LAND INCIDENTAL TO HIGHWAY DEVELOPMENT

BY THE DIVISION OF CONTROL, PUBLIC ROADS ADMINISTRATION

Reported by **H. R. BRIGGS**, Field Investigator and Statistician

INCREASING INTEREST has been directed in recent years to the problem of providing adequate rights-of-way for streets and highways. The construction of express highways and freeways to serve expeditiously large volumes of motor-vehicle traffic has become a serious problem in many of the more densely populated areas of the United States. The widening of existing routes as well as the adequate development of new routes is frequently delayed by the excessive costs of land acquisition, particularly in highly developed areas.

In early highway development the cost of rights-of-way was usually only a small percentage of the total expenditure and land acquisition was not a serious problem. This was particularly true in the development of the rural road system in some parts of the country where rights-of-way along section lines had been established by statute or acquired without cost through long uninterrupted public use. Today the development of main routes in rural areas frequently necessitates widened rights-of-way or the acquisition of broad, new rights-of-way. The improvement of highway connections in urban areas can frequently be accomplished only by widening existing streets or cutting new streets through highly improved areas. Since the right-of-way cost sometimes represents a high percentage of the total cost of the improvement, it becomes an increasingly important obstacle delaying modernization of streets and highways.

The modern highway is no longer merely a "strip of land bounded by approximately parallel boundaries for the purpose of direct travel" (*1*).[1] Heavily traveled main highways should have neutral strips separating the lanes for traffic in one direction from the lanes for traffic in the opposite direction. Supplementing these lanes for through traffic, one-way local-service roads may be provided on either side of the arterial lanes and separated from them by neutral strips. Within the right-of-way limits, provision may also be made for sidewalks and bicycle or bridle paths, in addition to public service facilities such as water mains, sewers, gas mains, and electric and telephone wires.

An important part of the right-of-way problem is the acquisition or control of land outside the immediate boundaries of a given improvement. Such acquisition is generally desirable from the standpoints of appearance, safety, economy, and the general public welfare. The taking of additional land may also facilitate the future development of a highway, particularly where widening may become necessary because of increased traffic. It has also been suggested that since increased property values often accompany a public improvement, measures should be taken to conserve this unearned increment to the public so as to defray in part the cost of the improvement. The conservation of these increased values may be accomplished through acquisition of excess land at the time of the improvement with subsequent resale after its completion. The acquisition of these excess lands either for marginal control or recoupment is commonly referred to as "excess condemnation."

The individual elements of "excess condemnation," namely, the necessity of marginal land[2] acquisition for adequate highway development, the varying possibilities and inherent limitations of recoupment acquisition, the increment tax and special assessments, and the analysis of increment and its relation to recoupment, have been made the subject of the present study. This report, which summarizes experience to date with various land-taking procedures in the United States and elsewhere, and discusses the limitations and possibilities of these elements in regard to present highway development and to adequate provision for present and future traffic needs, has been prepared at Madison, Wis., under a cooperative agreement between the University of Wisconsin and the Public Roads Administration.[3]

MARGINAL LAND ACQUISITION IS BEST SOLUTION OF REMNANT PROBLEM

Although the term "excess condemnation" might be construed to indicate that unnecessary or surplus land was acquired, usually only such land is taken as is necessary to make the improvement effective, or in some instances to avoid payment of high consequential damages. In addition, control of the development of land outside the physical limits of the highway is often essential if the maximum usefulness of the improvement is to be realized. Therefore, all land acquired outside such limits is not necessarily "excess" land. Rather than "excess condemnation," which has been loosely applied to the acquisition of land outside the boundaries of a street or park development, two terms may be used which convey a more nearly correct meaning. These are "marginal acquisition" and "recoupment acquisition."

Marginal acquisition is defined as the acquisition of land outside the boundaries of an improvement not directly required for its physical location but having a definite utility in connection with such improvement.

Recoupment acquisition is defined as the acquisition of land outside the boundaries of an improvement acquired for the purpose of resale so that some of the increment in land value resulting from the improvement will be recovered to pay part of the cost of the development.

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[1] Italic figures in parentheses refer to bibliography, p. 117.

[2] In this report the term "marginal land" is used to denote land bordering on highway rights-of-way.

[3] This work is under the direction of Dr. H. R. Trumbower, Senior Agricultural Transportation Economist. Acknowledgments of assistance in the preparation of this report are due David R. Levin and Ann R. McGinley.

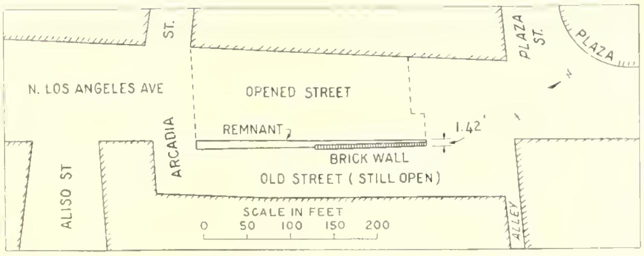

Figure 1.—Land Remnant Left When N. Los Angeles Avenue Was Opened, Los Angeles, California.

Of the advantages that may be obtained by marginal land acquisition in connection with street and highway development, the more important ones discussed in this article are: (1) Solution of the remnant problem; (2) elimination or reduction of consequential damages; (3) reduction of damage costs and of land costs for future development; (4) protection of the highway appearance and conservation or development of adjacent property values; (5) removal of traffic hazards; and (6) conversion of main, through highways into freeways.

Marginal land acquisition appears to be the best solution to the remnant problem, created when the opening or widening of a street disorganizes the land pattern of city streets, lots, and blocks.

When a thoroughfare is widened or cut through a developed area, often only parts of some lots or tracts are required for the improvement, and remnants or gores are left. Frequently these remnants are small and of irregular shape. In many instances these pieces of land are too small to have any substantial economic value, and are often used for the erection of signboards, shanties, or some type of stand. Hence, they are often detrimental both to the improvement itself and to adjacent property values when the remnants cut off access to property which otherwise would have frontage on the improvement.

Sometimes these remnants can be combined with adjacent properties or with one another. Since such combinations usually require lengthy negotiations which are especially protracted in the event of joint or absentee ownership, concurrent acquisition by the public of both essential properties and remnants may simplify negotiations in subsequent sales or trades.

The remnant problem is intensified when an existing building is partially destroyed in connection with a street or highway improvement. A striking example occurred in Los Angeles, Calif., where a remnant several hundred feet long, about 8 feet wide at one end, and 2 feet at the other end, was left in the widening of North Los Angeles Avenue. Figure 1 shows the location of the building which was demolished, and a wall two stories high which remained on the north end of this remnant after the building was removed. The intrinsic value of the lot was destroyed by the street opening; there were heavy severance damages; and an unsightly use of the remnant resulted. However, because of legal limitations the city could not acquire this property for use in connection with the land it owned on either side (*2*). To prevent such occurrences in the future, the California Constitution was amended in 1928 to give to the public the power to acquire lands outside the boundaries of improvements by purchase or condemnation.

In 1915 the Committee of Taxation for the City of New York, in a letter transmitting a report on excess condemnation (*3*), stated that New York furnished several horrible examples of remnants caused by cutting streets through built-up sections. One example cited was the Flatbush Avenue Extension. Three years after this improvement had been completed, despite its favorable location, development of the abutting property had not yet begun. At that time the street still looked "as if it had been devastated by an earthquake." (*3*). The State of New York adopted a constitutional amendment in 1913 permitting excess condemnation, and through legislative action in 1915 the power of excess condemnation was extended to the City of New York.

The effective use of this power was illustrated in the acquisition of land remnants in connection with the extension of Sixth Avenue. Five parcels in one block taken outside the right-of-way are shown in figure 2. Acquisition of this marginal property prevented the creation of irregular plots which might have been left occupied by partly demolished buildings. In one instance, there would have remained the remnant of a six-story building 95 feet long, 24 feet wide at one end, and running to a point at the other end.

LAWS SANCTIONING CONDEMNATION OF LAND REMNANTS HAVE GENERALLY BEEN DECLARED UNCONSTITUTIONAL

When Christie and Forsyth Streets in New York City were widened in 1929, land was taken from both sides of several blocks. Prior to the widening of the two streets the blocks between were 200 feet wide and fully developed. The combined takings necessary for the widening narrowed the intervening blocks to a width of 125 feet and demolished all existing buildings fronting on the east side of Christie Street and on the west side of Forsyth Street within the area of the improvement. Because of the property damage which resulted, it was thought to be in the best interests of the city to acquire the entire blocks between Canal and East Houston Streets. The plan of this widening is shown in figure 3. The excess land acquired was later landscaped and converted into playgrounds and small parks.

The remnant problem exists also in rural areas. A highway development may cut through a farm or decrease the size of an orchard or truck garden to such an extent as to leave the remainder unsuited for agricultural purposes. This is especially true if a highway so severs a farm that a small parcel is separated from the main property, and the situation is aggravated by the separation of a farm house from its outbuildings or by the total or partial destruction of these buildings.

Attempts have been made through legislation to sanction the acquisition of land remnants by condemnation, but the courts, in those cases which have come before them, have generally declared such laws unconstitutional. As early as 1812 in New York, and 1817 in South Carolina, State legislatures, with a view toward correcting the remnant situation, enacted laws permitting cities, when only a part of a lot was needed for highway purposes, to acquire the entire tract by condemnation. In 1824 in the case of *Dunn* v. *City of Charleston* (*4*), the court found the South Carolina act unconstitutional; and in 1834 in the *Matter of Albany Street* (*5*), the New York Supreme Court declared the New York law of 1812 void. One of the bases for the courts' decisions was that if the principle of taking the land outside the limits of a highway were approved, it would apply equally to the taking of either a small area or a large tract.

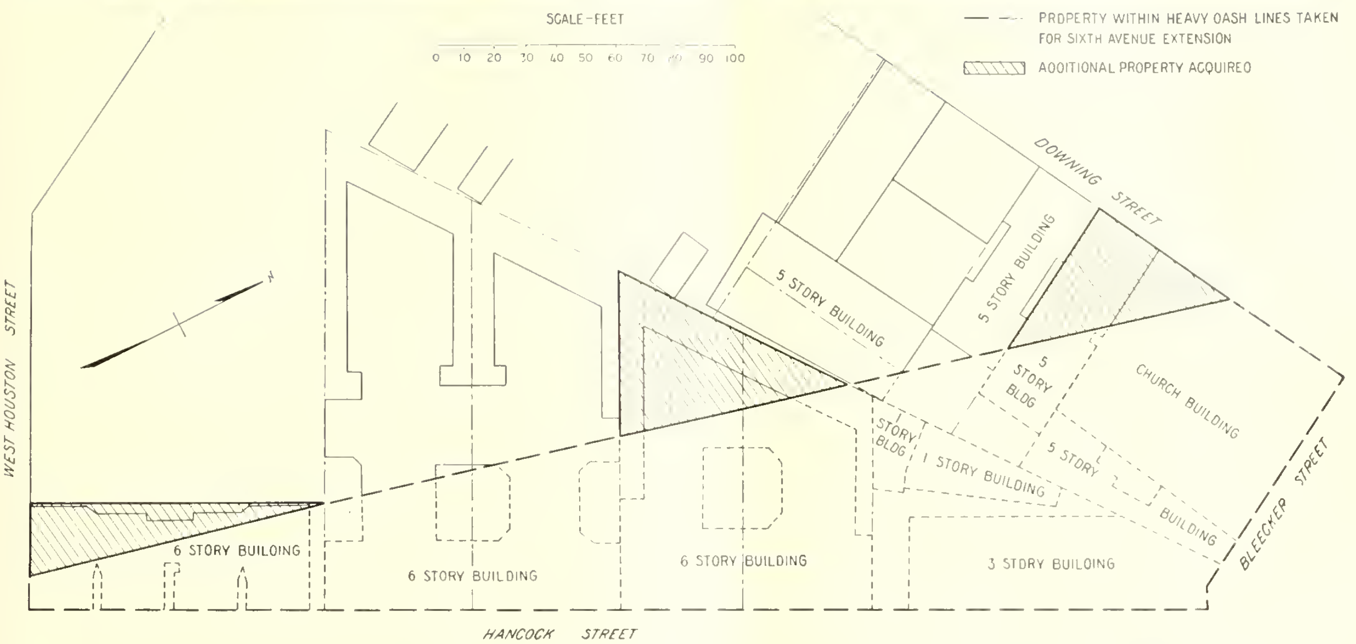

Figure 2.—Additional Property Acquired in Connection with Sixth Avenue Extension, New York City.

In *Embury* v. *Conner* (6), the doctrine of the Albany Street case was reaffirmed. Again in New York in 1863, in the case of *Bennett* v. *Boyle* (7), the court stated that in the widening of an avenue in a city, the powers and jurisdiction of the commissioners are limited and restrained to the lines of the avenue as enlarged. Beyond and outside those lines they can exert no power or authority whatever.

A case directly involving this point was that of the *City of Richmond* v. *Carneal* (8). A new street which cut diagonally across streets and blocks of the city was opened. The court held that the taking of land must be confined to the 80-foot strip needed for the right-of-way, notwithstanding that there was a State statute authorizing the taking of real estate abutting a proposed street when such property would be injuriously affected unless it were taken also and lot-lines readjusted.

In the five cases just cited where attempts were made under statutory authority to acquire land remnants in connection with highway improvements, the courts sanctioned the taking of only such land as was within the limits of the highway. However, remant acquisition was approved in a Massachusetts case. In *Opinion of the Justices*, (9), the Massachusetts Supreme Court indicated that if there were a land remant so small that a suitable building could not be erected upon it, then it would sustain the taking of such a parcel under the power of eminent domain. A Massachusetts law authorizing the taking of remnants was passed in 1904, but no case arose requiring a specific decision interpreting this authority. The *Opinion of the Justices* was given in 1910 in answer to a hypothetical question submitted by the legislature.

SEVERAL STATES HAVE ADOPTED CONSTITUTIONAL AMENDMENTS EXTENDING THE POWER OF EMINENT DOMAIN

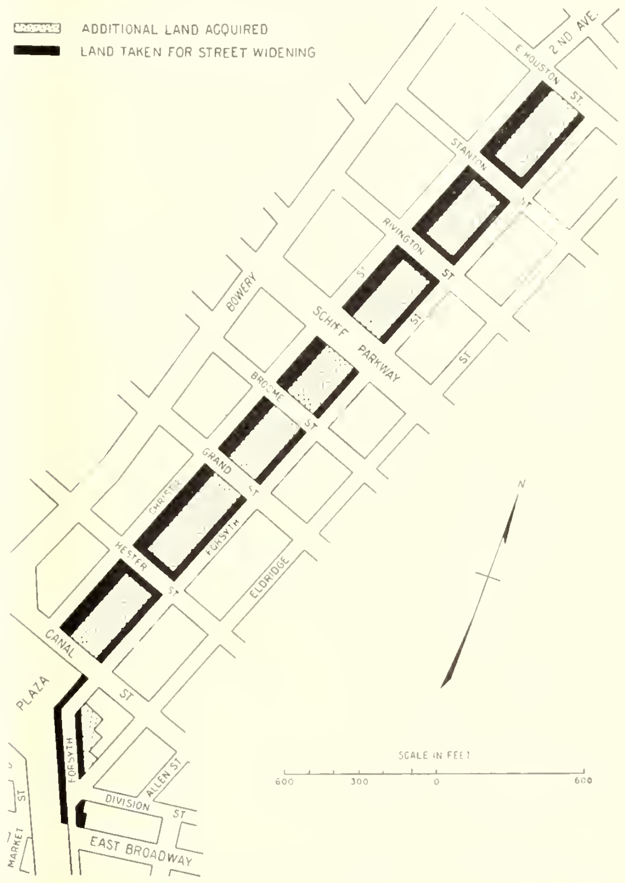

Figure 3.—Lands Taken in Christie-Forsyth Street Widening in New York City, 1929.

Constitutional amendments extending the power of eminent domain have been adopted in a number of States. Massachusetts in 1911 was the first State to extend the powers of eminent domain through constitutional amendment. Table 1 shows the dates of similar enactments in other States. The Virginia amendment transferred the power of determining the concept of "public use" from the courts to the legislature. The extent of the power granted varies in the different States, as can be seen from the following summary.

California, by article I, section 14½, of its constitution, authorizes the State or any of its cities or counties to condemn land

> in and about and along and leading to public works within 150 feet of a public work or improvement, provided that when parcels lie only partially within the 150 feet, such portions may be acquired which do not exceed 200 feet from the closest boundary.

Massachusetts, by amendment to article 10 of its constitution, permits more land to be taken by the Commonwealth, county, or city than is needed for the actual construction of the improvement, provided that no more land is to be taken than is needed for suitable building lots on both sides of the street.

Table 1.—*Dates of adoption of amendments to State constitutions extending power of eminent domain*

State	Date of adoption of constitutional amendment	Power extended to—
California	1928	State, counties, cities.
Massachusetts	1911	Do.
Michigan	1928	Municipalities.
New York	1913	Counties, cities.
Ohio	1912	Municipalities.
Pennsylvania	1933	Cities.
Rhode Island	1916	State, cities.
Virginia	1928	([1])
Wisconsin	1912	State, counties, cities.

[1] Allowed legislature to specify meaning of "public use."

Michigan, by article XIII, section 5 of its constitution, provides that municipalities may take land adjacent to an improvement that is appropriate for securing the greatest public advantage from the improvement. The surplus may be sold with or without restriction. Michigan is the only State with constitutional provision for recoupment condemnation.

New York, by article I, section 7 of its constitution, authorizes cities and counties to take more land than is needed for actual construction, provided that no more land shall be taken than is needed to form suitable building sites.

Ohio, by article XVIII, section 10 of its constitution, authorizes municipalities to take for public use property in excess of that actually occupied by the improvement. The surplus may be sold only with appropriate restrictions to protect the improvement.

Pennsylvania, by article XV, section 5 of its constitution, authorizes cities to take more land than is needed for actual construction but provides that the taking of land should "not be more than sufficient to form suitable building sites on such highways and streets." It further restricts use of this authority to streets connecting with bridges crossing streams or tunnels under streams which form State boundaries, to a point not more than 3 miles from such bridge or tunnel.

Rhode Island, by article XVII, section 1 of its constitution, authorizes the State, cities, and towns to acquire more land than is needed for public purposes, but confines the taking of lands to "no more than would be sufficient to form building sites abutting on such public highway, street, place, park, or parkway."

EXTENT OF AUTHORITY TO CONDEMN EXCESS LAND VARIES

Wisconsin, by article XI, section 3 of its constitution, authorizes the State or its cities to condemn lands in and about and along and leading to public works, and after the completion of the improvement, to sell the remainder with restrictions protecting the improvement.

In California, Michigan, Ohio, and Wisconsin, the authority is conferred directly upon the municipality to take the designated property under general laws. In Massachusetts, New York, Pennsylvania, and Rhode Island legislative action is required to authorize the use of the right to acquire additional property. Table 1 shows the units of government in the several States to which powers have been granted to acquire property outside the construction limits.

In addition to the several States granting through constitutional amendments the power to take land outside the construction limits of a highway, Delaware, Illinois, Indiana, Maryland, Nebraska, Oregon, and Virginia in 1938 had laws in effect broadening the scope of eminent domain powers. However, the constitutionality of some of these laws appears somewhat questionable.

In Delaware and Maryland, in specific instances, the courts have upheld the particular State laws, but have stated that it must be determined in each case whether the acquisition was actually for a public use. The following is a summary of the laws of the various States broadening the scope of the power of eminent domain.

The Delaware law (*10*), provides that a boulevard corporation may take as much more land as may be needed for the proper construction and security of the boulevard. The case of *Clendaniel* v. *Conrad* (*11*) approved the taking of a boulevard strip 200 feet wide, although but 30 feet were needed for the vehicular road.

The Illinois law (*12*) grants Chicago the right to take more land than is needed for an improvement when the court under which the condemnation proceedings are being held finds that such land is required to protect or aid the improvement and is of a reasonable quantity.

The Indiana Code (*13*) grants cities the right to take title to entire parcels when the damages are equal to the value of the entire tract.

A Maryland law (*14*) authorized the City of Baltimore to open, construct, and establish, a public highway over, along, and near Jones' Falls and to acquire property in and adjacent to the highway and incident to and for its construction. After completion of the highway, unneeded lands were to be sold and the proceeds placed in a Commission on City Plan Fund. The general principles of this law were upheld in *Duke Bond* v. *Baltimore* (*15*). The question as to whether there was an actual need for the taking of certain properties did not arise. Another Maryland law (*16*) granted special powers to the City of Pocomoke to take property adjoining markets, parks, squares, or other public places.

Metropolitan cities in Nebraska have the power (*17*) to take land in and about and along and leading to a street, park or similar enterprise, and to sell any surplus after the completion of the improvement. The law declares that such taking and conveyance constitutes a public use.

The Oregon law (*18*) gives municipal corporations the right to acquire property of a certain area immediately adjoining highways and not more than 100 feet distant.

The Virginia Code (*19*) empowers any city or town to acquire property adjacent to a street where the topography impairs the use of the thoroughfare. The power was also granted where a street opening or widening takes a part of a block or square, and, unless lot lines are readjusted, injuriously affects abutting property. Though this law was held unconstitutional in 1921 in *City of Richmond* v. *Carneal*, (*8*) a constitutional amendment in 1928 gave the general assembly the power to define "public uses." This appears to validate excess land acquisition in Virginia under the statutory authority.

CITY PRACTICES IN CONDEMNING EXCESS LAND DETERMINED BY QUESTIONNAIRES

To ascertain the extent to which marginal condemnation is used in cities, questionnaires were sent in 1938 to 155 cities having populations of more than 25,000 in those States which had adopted constitutional amendments extending the power of eminent domain. The following summary is an indication of the practices generally followed in the 81 cities from which replies were received.

1. Marginal land condemned for public improvements:
   - *a.* Seventy-four cities ordinarily take only the land that is actually and directly needed for the contemplated improvement.
   - *b.* Forty-seven cities never take land in excess of the amount actually and directly needed for a contemplated improvement.
   - *c.* Only 1 city (in Pennsylvania) reported frequent use of excess condemnation; 50 reported infrequent use; and 30 did not reply.
2. Land taken in excess of that actually and directly needed for a contemplated improvement.
   - *a.* Thirty cities take excess land to reduce suits and claims for severance damages.
   - *b.* Twenty-eight cities take excess land to prevent creation of small remnants of land.
   - *c.* Nineteen cities take excess land to remove present unsightly buildings so as to provide for landscaping future civic developments, etc.
   - *d.* Only seven cities take excess land to control the use of adjacent property through sale or lease to desirable tenants.
   - *e.* Only four cities take excess land with the objective of making a profit for the municipality through the sale of excess land which may become more valuable because of the improvement.
   - *f.* Twelve cities reported that excess land was taken to enable the city to defray part of the expenses of improvement by later selling the excess land, which will have acquired increased value because of the improvement.
3. Legal authorization.
   - *a.* Fifty cities reported that their legal authority permits possession and use of condemned property immediately upon payment of an agreed price or deposit of appraisal value and damages.
   - *b.* Twenty-three cities stated that possession and use of such property must await final determination by the courts.
4. Condemnation procedure.
   - *a.* Thirty cities reported that the condemnation procedure is easy and rapid.
   - *b.* Thirty-seven cities reported it slow and cumbersome.

The Committee on Right-of-way of the American Association of State Highway Officials in 1937 sent a general inquiry on right-of-way procedure to every State highway department in the United States. Replies to the inquiry were received from 39 States. One question concerned the disposition of excess land, and of the 39 responses received, 16 were inadequate or indicated that the respective States could not legally acquire property in fee. Some of the remaining 23 States reported their procedures, and others gave illustrations of their functioning under the legal authority they possess regarding excess land acquisition.

The 23 States which reported the procedure followed regarding acquisition and disposal of excess land are: Arizona, California, Colorado, Florida, Idaho, Indiana, Iowa, Kansas, Louisiana, Maine, Maryland, Massachusetts, Michigan, Mississippi, Montana, New Hampshire, North Dakota, Oregon, South Dakota, Utah, Vermont, Virginia, and Washington.

Although several of the States reported substantial financial benefits from the disposal of excess lands, not one reported the taking of surplus property with recoupment as an objective. The following paragraphs summarize the practices reported by the 23 States.

STATE PRACTICES REGARDING ACQUISITION OF EXCESS LANDS SUMMARIZED

Arizona considers it economical to acquire an entire piece of property when the highway traverses the major portion of it. Apparently the State retains these residual portions.

California sells or leases small pieces of land lying outside the main right-of-way when they are owned in fee and when such disposition is to the advantage of the highway commission. No excess real estate is acquired unless such acquisition will be to the financial benefit of the State.

Colorado, through its counties, purchases entire tracts when an owner is unwilling to sell a portion of a tract. Title to the land outside the right-of-way is vested in the county, and the county commissioners sell or lease it with restrictions as to its use.

Florida does not sell or lease extra-wide right-of-way. On minor relocations the unused portions are left to be reoccupied by the adjacent owners or may be quitclaimed by the county to the original owners.

Idaho sometimes obtains extra right-of-way in fee. Unused portions may be leased for agricultural purposes, but ordinarily revert to the adjacent owners when no longer needed for highway purposes. Occasionally some such land is quitclaimed to other parties.

Indiana sometimes obtains by grant land not specifically needed for highways. This is usually abandoned to adjoining owners, but where unneeded acreage is obtained by warranty deed, it occasionally is sold with the permission of the Governor. Additional right-of-way is sometimes retained for roadside development purposes.

Iowa through its executive council is permitted to sell, with certain restrictions and provisions, unused right-of-way upon recommendation of the highway commission. This has been definitely advantageous in facilitating settlements through exchanges or other transfers of right-of-way.

Kansas through its highway commission has authority by law to sell or exchange right-of-way. Occasionally a tract for which the State has no use is quitclaimed to the original owner, but none is conveyed to a third party.

Louisiana through its highway commission may acquire extra right-of-way in fee but not through process of eminent domain. When property is acquired by gift or purchase the highway commission may dispose of it at public auction or otherwise. A small profit has usually been realized on parcels that have been sold.

Maine purchases extra land through a warranty deed when it is necessary to do so to obtain the needed right-of-way economically, and this may be sold by the Governor and council upon recommendation of the highway commission. Land and buildings are sometimes rented until they can be sold.

Maryland sells the unused portions of any properties which have been purchased in the interests of economical land acquisition after public advertisement of such portions for sale. Dwellings upon such extra land are rented until sold.

In Massachusetts a special act of the legislature is required to sell or lease extra right-of-way.

Michigan sells excess property by deed or leases it by land contracts at prices determined by the State highway commissioner.

Mississippi occasionally acquires more land than is actually needed, but the highway department does not have authority to sell this extra property, authorization by the legislature being necessary.

Montana as a matter of expediency sometimes obtains more land than is actually necessary for right-of-way. Such excess land may be sold or, where it may be required for future development, a license to use may be issued. No land is leased or licensed for use for commercial purposes.

New Hampshire may purchase excess land by mutual agreement in acquiring right-of-way, and this may be sold by the Governor and council if recommended by the highway commissioner.

North Dakota permits reversion of extra or abandoned right-of-way which has been obtained through purchase or condemnation to the original owner or his heirs.

Oregon places control over all land acquired, including excess lands, in the engineering department, and all sales or leases must be recommended by the department engineers and approved by the State highway commission.

South Dakota has not sold and does not contemplate selling any extra right-of-way acquired.

Utah through the State road commission can sell or lease extra right-of-way under special circumstances.

Vermont permits the State highway board, with the consent of the auditor of accounts, to dispose of any land or buildings which are no longer of any use for highway purposes.

Virginia does not authorize the highway department to convey any real estate except by special act of the legislature. However, when it is to the advantage of the department, very often entire parcels are purchased under an agreement that the grantor simultaneously with the conveyance to the Commonwealth deeds the residue to parties designated by the highway department. This facilitates transactions and often enables the department to make adjustments with owners of adjoining land or to realize some return on an expensive piece of property.

Washington sells or otherwise disposes of any extra-wide sections or unused stock pile or pit sites when they are no longer necessary for highway purposes.

The acquisition of marginal or adjacent lands can also be of value in assisting adequate highway development by facilitating the sale or trading of property and by decreasing or eliminating consequential damages. When land remnants unsuited for a normal economic use are created, or when the utility of a residual tract of land is impaired by street or highway improvement, considerable amounts must often be paid for consequential damages. The public must pay not only for the land actually taken but also for damage to the remainder. However, the mere payment of these damages does not result in a right or interest in the property damaged, even though the total amount paid may approach or equal the actual value of the entire tract. If the acquisition of marginal land is possible, the public may obtain the entire tract, and although the total cost is not decreased or may even exceed the cost of the essential portion plus damages to the remainder, the public still has an advantage in that it acquires title to the entire property. It is then possible to combine and sell portions outside the right-of-way and thus recover some of the original cost; the excess portions may be traded to consummate transactions that might otherwise involve considerably more expense; or the remnants may be developed as small parks or planted areas.

MARGINAL LAND ACQUISITION USEFUL IN REDUCING CONSEQUENTIAL DAMAGES

In connection with the recent Kilbourn Avenue development in Milwaukee, Wis., it was necessary to acquire a lot occupied by a filling station. During other land acquisition on the same street the city was able to obtain at a favorable price an additional quantity of land which, after the new boulevard was developed, was a suitable site for a filling station. The city was able to trade this new location to the oil company as part of the consideration for its former site. The transaction resulted in a substantial decrease in the net cost of the improvement to the public.

An example in Elmhurst, Ill. may also be cited. For highway improvement in that vicinity a portion of a tract of land held by the city for potential park purposes was required and also another area essential for the city's sewage disposal plant. In connection with highway right-of-way acquisition, Du Page County was able to obtain a large tract of suitable land in the same general location. It then traded this single large tract to the city in exchange for the park and sewage plant properties which were needed for the highway.

A policy of marginal land acquisition intelligently applied to a highway development program may also make it possible to reduce excessive damage costs and eliminate land costs in future development of streets or highways. This is particularly true where reasonable forecasts indicate the need of future provision for increased traffic volume even though the present traffic volume or available funds may not warrant the full development immediately.

When a road is originally laid out, it may be located through relatively undeveloped areas where land costs are low. After it is established, development of contiguous areas may be encouraged and, if the road carries a large volume of traffic, ribbon development may set in. Business and residential sites become intermingled and commercial enterprises seek locations as close to the travel lanes as possible.

Subsequently, increased traffic may require a wider road. Adjacent property may have become more valuable and therefore large damage costs may be incurred because of development which the highway improvement itself originally encouraged. If, at the time of the original land acquisition, a sufficient quantity of land had been taken to provide for future development, these costs could have been largely avoided.

MARGINAL LAND ACQUISITION MAY PROTECT ADJACENT PROPERTY VALUES

A further advantage of a policy of marginal land acquisition is the opportunity it affords for protecting or improving the appearance of the roadway and thereby conserving values of adjacent property against unwarranted or unnecessary declines in value. This may be accomplished through public control which may be exercised by acquiring a right-of-way, either in the form of an easement or fee title, wider than that essential for construction purposes. Billboards and unsightly structures can thus be eliminated and the extra space used for roadside planting or development. Such planted strips are not only pleasing to the traveler but also screen dwellings from the road.

Acquisition of extra land along the highway provides adequate space for screening purposes and at the same time moves the building line a greater distance from the highway so that the disadvantages of residential locations along heavily traveled roads are minimized. These screens in semirural and rural areas also may serve as snow fences and thus reduce snow-removal costs.

Some courts apparently consider that in condemning property for public purposes it may be proper to consider the esthetic element and to take land outside of the physical limits of the improvement for that purpose.

In 1907, Pennsylvania passed a law permitting condemnation of land within 200 feet of a parkway with the express objective of protecting the improvement. The land so acquired could later be sold with such restrictions as would fully protect the parkways, "their environs, the preservation of the view, appearance, light, air, health, and usefulness thereof." An attempt was made in 1913 to apply the power given in the law in connection with the construction of Fairmount Parkway. The Pennsylvania Supreme Court in the case of the *Pennsylvania Mutual Life Insurance Company* v. *City of Philadelphia* (*20*), in a strong opinion refused to sanction the taking of land when the question of resale to persons other than the original owner was involved. It may be, however, that the fact that the city before acquiring the land and a building on it had already completed negotiations for its resale to a third party caused the court to consider that fact primarily rather than the preservation of the appearance of the avenue.

Although the courts looked with disfavor upon this particular acquisition of land outside the boundaries of a street development, there are instances where the interpretation of what constitutes a street limit has been liberal. In *Clendaniel* v. *Conrad*, (*11*), the court decided that boulevard corporations had the authority to condemn a strip of land 200 feet wide, although the vehicular road was only 30 feet wide. In *Valmont Development Co.* v. *Rosser* (*21*), it was held that land could be taken for an arnamental approach to a bridge and for future expansion. In *Duke Bond* v. *Baltimore*, (*15*), it was held that the city could take land on both sides of the development incident to and for the purposes of construction of the highway.

In Missouri, in the case of *St. Louis* v. *Breuer* (*22*), the court drew a distinction between a "road or street" and "parkway or boulevard." It said the word "avenue" does not have a general meaning which applies to both street and boulevard, but is synonymous with "street" and not with "boulevard." In the case of *Newbold* v. *Brotzge*, (*23*), the court defined a "parkway or boulevard" as a street of special width, given a park-like appearance by reserving spaces for trees, flowers, etc., and not used for heavy teaming; or one especially designed for pleasure walking or driving. Similar instances of this distinction may be found in the cases of *Bouis* v. *City of Baltimore* (*24*), and *Chaplin* v. *Kansas City* (*25*). Since one of the objectives in creating a parkway or boulevard is to create a thoroughfare having a pleasing and park-like appearance, it appears that the acquisition of land needed to accomplish such purposes might be sanctioned.

MARGINAL LAND ACQUISITION ENABLES ELIMINATION OF CERTAIN TRAFFIC HAZARDS

Since a main highway is a magnet for business enterprises catering to the motoring public, certain traffic hazards may be avoided or eliminated through acquisition of marginal lands. Such acquisition would permit control of the particular types of enterprise whose numbers generally increase with traffic volume, namely taverns, filling stations, and various types of roadside stands. Not only are hazards created by frequent interruptions of traffic flow, but these enterprises seek locations favorable to them but which often create confusion, particularly at night, at corners and curves because of advertising devices. If the public controls land along the highway, such hazards can be eliminated. To some extent, the same results can be accomplished through zoning; but since this method of control can be used only to prevent a potential objectionable use, outright public ownership is required to remove existing hazards.

Marginal land acquistion may also be especially useful in the development of freeways, or thoroughfares to which access is provided only at rather long intervals. One of the basic rights of property is that of access, which in most States is construed to apply to both old established ways and to newly laid out roads and streets. On freeways abutting properties are denied the right of access. By acquiring land outside the main traveled road local-service roads can be constructed, thus providing accessibility commensurate with that provided by the original highway.

Two constitutional cases which arose in Ohio, one of which was eventually carried to the United States Supreme Court, are of particular interest in connection with the possible uses of marginal land condemnation to facilitate desirable street and road development. These were the cases of *East Cleveland* v. *Nau* (*26*) and *Cincinnati* v. *Vester* (*27*). In both, the condemnation of excess lands by the city was held improper on the ground that the municipalities must define specifically the purpose for appropriating such lands, and must prove that the excess is reasonably needed in connection with the development. The court found that in neither case had the condemnation procedure followed been in conformity with that required by the authority given under the constitutional amendment passed in Ohio in 1912.

In the Nau case the city of East Cleveland passed legislation extending a street and provided that such extension should be 100 feet in width. The legislation also provided for taking an excess of land beyond that needed for the street, setting forth that the excess was needed for slope grading. Because of the peculiar topography, the city claimed that cuts would have to be made in defendants' lands and hence grading would be necessary. Pursuant to the legislation, the city filed applications to assess compensation to be paid for the taking of the 100-foot strip and also the excess.

The defendants in the case meanwhile filed petitions to enjoin the city from appropriating the excess land not actually needed for the improvement. They alleged that the property sought was in excess of that actually needed; that the appropriation was not for a public use but was for the purpose of reselling to private persons at a profit for the city; that the reason for taking the excess property was not properly defined in the city's legislation; and that for the above reasons, the proceedings by the city were void and contravened the provisions of the State and Federal constitutions, especially article XVIII, section 10 of the State constitution which provides that a municipality appropriating property for public use might "in furtherance of such public use appropriate or acquire an excess over that actually to be occupied by the improvement."

The Supreme Court of Ohio held that the city was required to define specifically in its legislation the purposes of its appropriation, and that it was incumbent upon the city to sustain such requirement by proof of its necessity since the power to condemn excess property is granted to the city only when the excess property is reasonably needed. It was also held that the larger part of the excess lands was not necessary for lateral support of the improvement and was not needed in furtherance of a public use, and that the grade so established by the city (which grade would have taken the whole of the defendants' properties) was wholly unreasonable and unnecessary for the maintenance of the street. The court held that it would not disturb a reasonable discretion exercised by city authorities, but that it could not sanction an arbitrary and unnecessary appropriation of excess land when the weight of evidence and testimony did not show that such excess land was needed.

The issues involved are succinctly stated by the Supreme Court of Ohio in its decision in the following language:

> The principal issue presented in these cases is this: Were all of such excess lands needed in the furtherance of this improvement or was their attempted appropriation arbitrary and needless? Upon that issue, both of the lower courts found that the appropriation of the larger portion of the excess lands was not necessary for lateral support for the improvement, and was not in furtherance of the public use. The lots of the several defendants in error, facing the improvement, were 130 feet in depth and incumbered by buildings. All of this property was sought to be taken for a proposed grade extending from the sidewalk, to be "graded on a slope which rises three-eighths of 1 inch per foot of horizontal distance from the grade of said sidewalk," etc. The grade so established by the city, taking substantially the whole of the properties of the several defendants in error, was wholly unreasonable and unnecessary for the maintenance of the street, or for its lateral support. The arbitrary fixing of a three-eighths of an inch grade therefore was unreasonable. This is evidenced by the testimony of one of the engineers introduced by the city, who in answer to the court's query, testified, in effect, that not more than 10 feet or 20 feet from the sidewalk would be necessary for grade maintenance.

In the Vester case[4] excess condemnation of the properties in question was proposed by a resolution adopted by the Cincinnati City Council in accordance with Ohio statues,[5] but the purpose of the appropriation was stated in the resolution only in general terms as being "in furtherance of the said widening of Fifth Street" and "necessary for the complete enjoyment and preservation of said public use."

U. S. SUPREME COURT REFUSED TO SANCTION TAKING OF EXCESS LAND IN AN OHIO CASE

The ordinance providing for the excess appropriation was not specific, declaring simply that it was "in furtherance of the public use," described as the widening of Fifth Street, and "for the more complete enjoyment and preservation of the benefits to accrue from said public use." In what way the excess condemnation of these properties was in furtherance of the widening of the street, and why it was necessary for the complete enjoyment and preservation of the public use of the widened street were not stated and were thus left to surmise.

The city argued that in resorting to excess condemnation legislative bodies generally have had in view the following three purposes: (1) The avoidance of remnant lots; (2) the preservation and amplification of the improvement; and (3) the recoupment of expense from increased values. Both the district court and the circuit court of appeals concluded that the theory of remnants, and of the protection and preservation of the improvement, were not applicable. Both courts believed the sole purpose of the city was the recoupment of a large part of the expense of the street widening by the resale of the properties in question and that the excess condemnation was in violation of the constitutional rights of the plaintiffs because the condemnation was not a taking for a public use "within the meaning of that term as it heretofore has been held to justify the taking of private property." The court of appeals added that the provision of the State constitution relating to excess condemnation, "would seem to mean in furtherance of the normal use to which the property that is occupied by the improvement is devoted—here the use and preservation of the street for the purposes of travel," and the court held that if the provision means that property may be taken "for the purpose of selling it at a profit and paying for the improvement it is clearly invalid."

In the United States Supreme Court, the city challenged the propriety of the assumption that the city was proceeding on the theory of the recoupment of expense by resale of the properties, and insisted that its purpose in the specific cases could not thus be delimited. The city urged that, when the improvement was completed, the city council would doubtless be in a position to determine what sized tracts and what kinds of restriction would be best suited for the harmonious development of the south side of the street. The city also insisted that it might never resell the excess; that it was not compelled to do so by the constitution; that the question was one to be determined in the future; that recoupment could come only from a sale; and that until by some act the city evidenced an intent to sell, it could not be said to be proceeding only on a theory of recoupment. After a further consideration of whether the purposes of the excess condemnation had been adequately stated or not, and a discussion of the judicial or legislative sphere of deciding what was a proper public use, the court held that questions relating to the constitutional validity of an excess condemnation should not be determined upon conjecture as to the contemplated purpose. The court held that the object of the excess appropriation had not been set forth as required by the local law, and concluded that the proceedings for excess condemnation of the properties involved were not taken in conformity with the applicable law of the State.

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[4] For a digest of this case, see Toll Roads and Free Roads, 76th Congress, 1st session, House Document No. 272, pp. 126–128.

[5] General Code of Ohio, sec. 3679: Resolution shall be passed. When it is deemed necessary to appropriate property, council shall pass a resolution, declaring such intent, defining the purpose of the appropriation, setting forth a pertinent description of the land, and the estate or interest therein desired to be appropriated. For waterworks purposes and for the purpose of creating reservoirs to provide for a supply of water, the council may appropriate such property as it may determine to be necessary.

In affirming the decrees made by the lower courts, the Supreme Court concluded that the proceedings for excess condemnation of the properties involved were not taken in conformity with the applicable law of the State but refrained from expressing an opinion upon the other questions that were argued.

EXCESS LAND SOMETIMES SOLD OR LEASED TO REDUCE COST OF IMPROVEMENT

Various phases of marginal land acquisition have already been discussed. All of these phases were directly concerned with the utility of the improvement. However, marginal land may sometimes be sold or leased for the purpose of decreasing the cost of the improvement, but such transactions are always incidental to the primary objectives of the acquisition. When real estate has been obtained for any given specific improvement, if the title to the property has been acquired rather than an easement for the public use, the public may sell the property when it becomes evident that the land is no longer needed for the purpose for which it was originally acquired.

A noteworthy instance of such sale of land occurred in Boston in the case of the Back Bay flats. These low lands adjacent to the Charles River were usually flooded at high tide, and were not only unusable and prevented proper development of the harbor, but were a decided menace to health because of their stagnant condition. About 60 years ago the area was condemned, drained, and filled, and after it was platted and sold, became the well-known Back Bay residential district.

The Massachusetts Supreme Court in the case of *Moore* v. *Sanford et al.* (*28*) sustained this sale on the grounds that it was incidental to the primary object of the acquisition which was to remedy a condition that was detrimental to the welfare of the city.[6]

In connection with the construction of the Center Street subway New York City acquired some extra land along Canal Street. The city sold the adjacent property acquired "subject to the easement and reserving the space needed for entrance, at a price so near the original purchase price that the easements and the space for station entrances involved almost a nominal expense" (*29*).

Unfortunately, in many highway improvements a barrier exists to the disposal of surplus land because the property interest acquired is an easement for highway purposes rather than the fee to the property. All rights, therefore, are contingent upon the public continuing to use the property for highway purposes. When title to land is vested in the public, it may sell or otherwise dispose of such property if the road or street is closed or abandoned; but if the public right is only an easement for highway purposes, such property may revert to the original owners when use of the property for highway purposes ceases. As far as cost is concerned, normally there is little difference between the price commanded for full title and that paid for an easement. Under these conditions, it is in the public interest to acquire title rather than an easement when property is taken for highway purposes.

In contrast to the sale of incidentally acquired land, property may also be acquired specifically for the purpose of resale under a public land fund or authority so that the proceeds may be used to reduce the cost of the improvement. Ordinarily, a governmental organization may go into the market as any private individual and purchase land. Governmental participation in land enterprises for the public good is an old and well-established practice. For example, Glasgow, Scotland, has had a Common Good Fund for 350 years. This is defined as consisting "of such property and funds as are held by the magistrates and council on behoof of the community, unfettered by restriction as to its disposal, save conformity to the common law and the promotion of the public weal of the burgh." In 1927, this fund had assets of approximately $50,000,000, and liabilities of approximately $47,500,000. The street railroad system had been bought and paid for out of this fund (*30*).

The German cities of Ulm and Düsseldorf have also reported successful use of land funds. Düsseldorf had a special land department financed by a bond issue supported from a ¼-mill tax levy. The business of the department was to invest in land for the city and to anticipate future wants. When a city department needed a piece of land held by the land fund, the lot was transferred and its value paid over to the fund (*31*). Common examples of land funds in the United States are those used in many States for school purposes. In 1938 Pasadena, Calif. had a Land Sale and Purchase Fund which permitted the purchase of land without condemnation, and Toledo, Ohio, had a Permanent Improvement Fund which could be used for purchasing property needed for municipal purposes only.

VALUES CREATED BY PUBLIC IMPROVEMENTS RETAINED THROUGH RECOUPMENT CONDEMNATION

A third method by which the cost of public improvements may be reduced through the sale or lease of property favorably affected by the development involves the process of recoupment condemnation under the power of eminent domain. In some instances a public organization, in an effort to retain for the public the full increment created by a public improvement, may attempt to use its sovereign power of eminent domain to take benefited properties forcibly with the objective of selling the property so acquired at the enhanced value.

Recoupment condemnation has been used to some extent in Europe. Figures have been given by various writers showing the ostensible results of recoupment condemnation in European cities. The greatest single enterprise and the one most commonly cited is the Strand-Holbern or Kingsway development in London. Various figures have been given concerning this project, but an authentic statement of the financial results is to be found in a report to the London County Council by its Highway Committee, presented December 15, 1936 (*32*). This report, in part, states:

> The council receives about $695,910 annually in ground rents from the surplus land of the improvement, upon which buildings of the approximate cost of $24,332,500 have been erected. In addition, a sum of $4,701 a year is received in respect of better-

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[6] The taking of these lands and flats was authorized by the statutes of 1884, c. 290. Such legislative action was made necessary by the reluctance of certain owners to sell their property, or to join in the proposed improvement.

> ment charges. A total sum of $3,579,345 has been received from the sale of the sites, and other sites valued at $1,096,379 were transferred in settlement of claims on reinstatement basis.
>
> The total debt charges incurred to March 31, 1936 ($25,352,338), plus the net debt outstanding at that date ($15,614,686), amounted to $40,967,024. Against this, the aggregate rents received, plus the value of the leased sites, amounted to $29,247,329. The difference ($11,719,695) may, therefore, be said to represent the net cost to the rate payer up to March 31, 1936.
>
> The annual net charges on the rates (that is, the total debt charges less the rents and improvement charges), which in 1935–36 amounted to $295,650, will be reduced as loan charges fall out. In the year 1955–56, it is anticipated that there will be a small surplus of $9,733. This and ensuing surpluses will be a credit to rate account year by year. A large part of the debt will be retired in 1961–62, so that the total debt charges in that year will decrease by about $555,642. For 1965–66 the surplus for the year is estimated at $699,438, increasing later to $710,509, the debt being finally paid off in 1987–88.[7]

Before general conclusions can be drawn, however, as to any results obtained through recoupment, five elements should be carefully investigated and considered: (1) Actual amounts used as costs and recoveries, (2) extent of the use of condemnation, (3) number of parcels involved, (4) types of property taken, and (5) nature of the improvements.

A study of these examples requires a careful analysis of the costs. When reference is made to an "excess condemnation" enterprise, a certain amount is usually reported as the original cost to the public, and another amount as the sum recovered. The cost reported may be only the amount paid to the property owners and may fail to include such items as real estate commissions and court costs, which are usually high in condemnation cases. The total outlay which should be offset against the total recoveries from the sales of property may be considerably more than the reported original amounts paid to property owners.

Another financial element which needs examination is the amount received for the property. Each enterprise should be studied to see how much property has actually been sold and how much still remains in the hands of the public. For example, it may be said, as a result of a certain condemnation transaction, that the city received $100,000. A further investigation of this amount may disclose that the city has perhaps received only $50,000 in cash and has on hand land which has been estimated at a value of $50,000. The yield from a recoupment enterprise should not be considered as final until disposition has been made of all the property acquired.

IMPROVEMENTS MAY CREATE NEW VALUES OR CAUSE SHIFT OF VALUES

The extent to which condemnation is actually used should be determined in analyzing the results on various improvements. In the Fairmount Parkway Improvement in Philadelphia, it was necessary to acquire approximately 1,000 parcels of land. All but three of these were obtained through direct negotiation. Most of the land in many enterprises may thus have been acquired through normal acquisition, and condemnation resorted to only in obtaining a few remaining tracts. It is incorrect to credit the success of undertakings to excess condemnation in those cases which involve condemnation only incidentally. The financial success may be credited to the land obtained by purchase rather than to that taken by condemnation.

An analysis should also be made of the number of parcels involved in condemnation undertakings. Since there are certain definite charges to be met in connection with each individual acquisition, such as title search, recording, and appraisal fees, the acquisition of a few large tracts requires less incidental expenditure than the acquisition of numerous small parcels. This is particularly true in condemnation cases, where court costs must be added to the other incidental expenditures.

An adequate analysis of the value of property acquired by condemnation before and after the improvement is also required. If the dominant utility of the land is materially increased by the development, a substantial increase in value may result. Examples are the Northumberland Avenue opening in London in 1876, and the Michigan Avenue widening in Chicago in 1920. In the Northumberland Avenue project, a new, short street connecting two business areas was created. The territory traversed was unimproved. All the land appears to have been bought from one owner. Land formerly vacant was turned into good business sites and a profit of approximately $595,000 was made through the sale of excess land (*3*). In the case of Michigan Avenue, a narrow street lined by warehouses, loft buildings, and wholesale establishments, became a major artery suitable for a high class, retail business section. An intensive new development occurred with a temporary maximum appreciation of 2,500 percent over the former value (*33*).

Finally, a study of the nature of the improvement is also essential since the character of the improvement itself can affect the extent and value of the increment. The development may consist either of improving existing facilities or creating entirely new ones. For example, a highway project may consist of widening or improving an existing road or street, or it may consist of cutting a new highway through an area where normal development has been hampered through lack of transportation facilities. Where a new facility is created, an increase in land values may be expected.

The London experience demonstrates that a greater percentage of the cost of the improvement can be recovered when a new street is opened than when existing thoroughfares are widened. In street improvements made by the Metropolitan Board of Public Works of London, the recoupment obtained where existing thoroughfares were widened was 24 percent of the gross cost and 32 percent of the net cost of the undertakings, whereas in the construction of the new streets, 48 percent of the gross cost and 88 percent of the net cost was recovered. (*3*)

Of the three methods outlined above, the first (sale or lease of property acquired incidentally) and the second (sale or lease of property acquired under public land funds or authority specifically for the purpose of resale) do not involve the principle of recoupment condemnation. The use of this term should properly be limited to the third method. While most authorities on the subject carefully make this distinction, some writers have referred to recoveries made under incidental sales or through the operation of a public land fund as examples of the savings made through "excess condemnation." For example, in an analysis prepared for the Wisconsin Legislature, (*34*) the operation of the public land funds of Ulm and Düsseldorf are cited as demonstrations of what can be done under the scheme of excess condemnation.

Experience indicates that the difficulties inherent in condemnation are such as to render its use impractical when many properties are involved. Condemnation may be a lengthy and tedious process and litigation may last for years. In Milwaukee, land acquisition for the Kilbourn Avenue development was started in April 1932 under the Kline law. However, the legality of the proceeding was questioned, and it was not until December 1938, that the Wisconsin Supreme Court gave the decision sustaining the procedure.

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[7] The dollar values shown are based on the pound value at $4.8665.

Another limitation is the fact that apparently high, if not exorbitant, awards are frequently given by juries in condemnation cases. In certain improvements made in Paris between 1852 and 1869, where condemnation, or compulsory taking, was used, the city recovered only about one-fourth of the amount it originally paid for the land. This was attributed largely to the high awards allowed by the jury.

Experience with the London improvements indicates "that owners of property are tempted to make arrangements in the way of new lettings or improvements, as soon as it is known that an improvement is contemplated along their holdings, so as to fleece the city. In some instances, the money thus exacted has enormously increased the cost of the work." *(3)* A summary of the results in various recoupment acquisitions is given in table 2.

TABLE 2.—*Summary of reported costs and recoveries in selected recoupment enterprises*

Nature and location of improvement	Date	Total cost	Amount recovered	Net cost	Percentage recovered
Baron Hausmann Blvd., development of 56.25 miles of improved streets, Paris [1]	1852–69	$259, 400, 000	$66, 200, 000	$193, 200, 000	25. 5
26 streets widened by Metropolitan Board of Public Works, London [2]	1855–99	26, 375, 820	6, 409, 290	19, 966, 530	24. 3
16 new streets, London [2]	1855–99	35, 117, 265	16, 722, 365	18, 394, 900	47. 6
3 Thames embankments, London [2]	1855–99	76, 797, 445	26, 230, 530	50, 566, 915	34. 2
16 slum clearances, London [2]	1876–84	8, 349, 995	1, 755, 030	6, 594, 965	21. 0
43 highways, 1 slum clearance, London [2]	1889–1913	44, 246, 125	23, 511, 740	20, 734, 385	53. 1
St. Lawrence Blvd. extension, Cartier St. opening George Etienne Cartier Sq. development, Montreal [1]	1912	872, 728	927, 325	−54, 597	106. 3
Devonshire Court opening, Rochester, N. Y. [3]	1919	17, 224	18, 750	−1, 526	108. 9

[1] Robert E. Cushman, Excess Condemnation.

[2] *(3)*.

[3] A. L. Veddor, The Use of Excess Condemnation in the Opening, Widening, and Extension of Streets, Proceedings, American Society of Civil Engineers, Sept. 1925, p. 1420.

INCREMENT TAX MORE WIDELY USED IN EUROPE THAN IN THE UNITED STATES

One of the means of conserving for the public good some of the increased value created by public improvements is the increment tax. This tax upon the increase in land value may be imposed in two ways. It may be imposed periodically or at the time of transfer. When the tax is imposed periodically, valuations are made at stated intervals. Under the other plan a tax is levied at the time of a sale, lease, or inheritance. This method of taxation is quite common in Europe. It differs from a special assessment in that a special assessment is a charge made to defray the cost of a particular improvement. The increment tax is based on the theory that, either through a specific improvement or through general development, an unearned increment in value has been realized, and that this increase should inure to the public.

While income and inheritance taxes in the United State may be considered to a certain extent as increment taxes, there are basic differences in the European and American plans of taxation that cause the increment tax on real estate to be more suitable for European countries than for the United States. The primary difference is that in this country the basis for general taxation is the value of property, while in Europe the basis is the income from property.

Extensive recovery of the cost of improvements in the United States is sometimes obtained by use of the special assessment principle, which is peculiarly an American doctrine. While the use of special assessments is subject to grave and legitimate criticism, it has proved a practical way of collecting from the property owner a part or all (and sometimes even more), of the increment in the value of the property resulting from the public improvements. A few examples will illustrate returns to the public obtained through special assessments

For the fiscal year ending June 30, 1930, the city of Detroit received $10,425,458 in benefit charges *(35)*, of which $6,925,500 *(36)* was for highways. In 1930, in Wayne County, Mich., 8.5 miles of the Schoolcraft 204-foot superhighway, known as Assessment District Road No. 11, and 9 miles of the 120-foot Inkster Road, known as Assessment District Road No. 14, were built and financed largely through benefit charges *(36)*. Wayne County, in 1930, proposed a superhighway plan in which the right-of-way would cost $100,000,000. Of this cost, $20,000,000 would be defrayed by special assessments *(37)*.

In Chicago, in the year 1930 alone, the city's net bills to property owners totalled $34,504,100 for benefits resulting from previous street developments, and in addition a large part of its current highway costs of $75,895,800 was defrayed through street improvement bonds *(38)*. During the 10 years from December 31, 1922, to December 31, 1932, the total special assessments recorded in the city of Chicago amounted to $268,908,825 *(39)*. Herbert D. Simpson has estimated that in the 10 years preceding 1930 the people of the United States had spent approximately $7,000,000,000 on public improvements in the form of streets, highways, parks, and reclamation and irrigation projects; that of this cost approximately $2,500,000,000 was levied in the form of special assessment charges upon the land affected; and that the cities in the United States of 30,000 population or more had spent more than $2,000,000,000 on highway improvements alone and more than half of this was financed in the form of special assessments on the land affected *(33)*.

The park system of Kansas City, Mo., up to June 30, 1930, cost $25,500,000. This park system was financed by special assessments *(40)*.

The figures presented are taken from investigations of special assessments on improvements inaugurated during the period 1920 to 1930 which may well be considered the period of greatest use of this method of defraying public costs. However, a large portion of public costs is still being recovered through special assessments. For instance, New York City issued $44,700,000 of assessment bonds for street improvements and street and park openings from December 31, 1938, to November 24, 1939 *(41)*.

Michigan from January 1, 1935, through 1938 floated 24 new bond issues for inter-county Covert roads. Of the total amount issued, $2,324,872 was still outstanding in 1938, a portion of which was a liability of special assessment districts. Though the liability of the assessment districts varies from 20 to 100 percent for each of the several bond issues, actually allocations of State receipts from the automobile registration and gasoline taxes are used to pay debt interest and retirement, thus relieving practically all of the property owners in the special assessment districts of this burden (*41*). Likewise, on the Kilbourn Avenue development in Milwaukee, the total assessable cost was estimated to be between $3,926,000 and $4,024,000 as of December 1938 (*42*).

Although these examples show that special assessments are still being used quite generally and effectively to conserve to the public some of the value created through public enterprises, use of this method of recovery is not as general as it was about 1930. Local expenditures for public improvement have been substantially curtailed in the past decade, and a large amount of local expenditures has been replaced by FERA, PWA, and WPA funds.

PUBLIC IMPROVEMENTS DO NOT ALWAYS INCREASE ADJACENT PROPERTY VALUES

It is generally believed that public improvements tend to increase adjacent property values. For instance, along California State Highway 26 east of Los Angeles, a strip of lots was offered for sale in 1934 at $200 to $250 per lot. After improvement of this traffic artery, these same lots, less a 30-foot strip which was donated to the State, sold at $500 to $1,500 per lot (*43*).

In Chicago, the Blue Book of Land Values indicates that residential property on boulevards is worth twice as much per front foot as property only a short distance away. A spectacular increase in land values followed the widening of Michigan Avenue in Chicago. "While the initial Michigan Avenue improvement cost $16,000,000 it added $100,000,000 to property values" (*33*).

A memorandum of the City Club of New York of October 2, 1908, to the Board of Estimate and Public Service Commission stated that from 1900 to 1910 the land from One Hundred Thirty-fifth Street to Spuytin Duyvil increased $49,200,000 in value due to the construction of the subway, while the entire cost of the subway was but $43,000,000 (*44*).

Such examples have led to the erroneous assumption that all public improvements directly cause an increase in value of adjacent property. However, it is only when an improvement is well planned and when adjacent property has not been intensively developed that a public improvement may create or stabilize values in its vicinity. An apparent increase or decrease in values may upon analyses be found to consist of a shift of values from one locality to another.

Statements concerning the effect of public improvements on land values are found in the writings of various authorities. Ely and Morehouse have stated (*45*):

> The effect of a city's expenditures for public improvements, such as streets and parks, is to strengthen a rising movement of land values established by other causes. Independently of other factors, public improvements ordinarily do not cause increased values, but when joined with other influences they have an effect in that direction.

A study of land values in New York City in relation to transit facilities was made in 1930 by Edwin H. Spengler. His study was prompted by the assumption of various commissions and public bodies that extension of rapid transit facilities directly and inevitably increases the value of urban land. After an exhaustive investigation, he came to the following conclusions (*44*):

1. The building of subways in New York has been accompanied by shifts in land values from one part of the city to another; values have been transferred rather than increased.
2. Certain influences upon land values have frequently caused decreases which have exceeded the increases resulting from the opening of new transit facilities.
3. Centers of population tend to form and grow, thus enhancing the value of such centers at the expense of other areas.
4. Land along transit routes changes in value depending upon the character of growth of the whole area through which the routes pass. These values rise substantially in regions that show rapid expansion, change little in somewhat "settled" areas, and drop in regions that undergo a general decline.
5. A development stimulating the demand for land in a certain locality will have the effect of increasing land values. This may take place during the construction of a transit line, directly after its opening, or many years after its construction. The increase may even occur in a territory not directly supplied with special transit facilities but accessible from regions so supplied.
6. Instead of being considered a cause of land value changes, a transit line should more properly be regarded as a facility which permits or fosters, under certain circumstances, an emergence of land values, the values being determined largely by other factors.

The same general conclusions were reached by Herbert D. Simpson (*33*). He has stated that increases in land supply through increased accessibility, availability, and usability because of transportation development, public improvements, and other factors most normally tend to decrease total land values; that is, these factors tend to abstract from existing values more than they add in the form of new land values, but that this effect has largely been concealed by a general appreciation of land values during the last two or three decades. He cites his investigation of the Michigan Avenue development and states that this improvement created an appreciation in value of $104,290,000 on the west side of Michigan Avenue from Randolph Street northward to Pearson Street and was accompanied by a depreciation of $609,198,420 on the west side of Michigan Avenue from Congress Street southward to 31st Street.

Examples previously cited show that normally there may be an increment of value in connection with a highway development. To a certain extent, the relative amount of this increment will vary inversely with the length of the improvement. That is, as the length of the development increases the percentage of value increment decreases.

A major highway development may open more territory than can be absorbed immediately and lead to premature and uneconomic subdivision. Although an increment of value may result from a public improvement, and although the public can conserve this increment for itself either through the acquisition of excess land, the operation of a public land fund, or the imposition of special assessments, it can do so only after careful planning and investigation.

SUMMARY

Increasing interest is being directed toward acquiring wider rights-of-way for highways. This is primarily due to the fact that necessary improvements can frequently be accomplished only by cutting through highly developed areas. Considerable thought should be given to the various methods of acquiring land so that maximum usefulness and utility, together with provision for future needs, may be obtained from any given improvement. This article has attempted to show, by distinguishing between marginal and recoupment acquisition, the effect of various procedures which may be followed in land acquisition and their advantages and limitations, as well as to show means by which the public may conserve for itself any increment in property values resulting from the improvement.

A consistent policy of marginal land acquisition may be effective: (1) In solving the economic and general community problem of handling land remnants created as a result of street or highway improvement; (2) in eliminating or reducing consequential damages; (3) in preventing excessive damage costs in present and future development; (4) in conserving property values and protecting public improvements; (5) in avoiding traffic hazards; and (6) in facilitating the conversion of main through routes into freeways.

Recoupment may be realized through the sale or lease of incidentally acquired property or the sale or lease of property specifically acquired under public land funds or authority. While recoupment acquisition when used in connection with the power of eminent domain may actually result in lowered costs for the improvement, such acquisitions should not be used indiscriminately since the success of the method depends upon the extent of the condemnation, the number of parcels involved, and the nature of the improvement.

Although several States have constitutional authority which permits them to acquire excess lands under the power of eminent domain, and a number of States have statutory authority, investigations show that this power is being used only to a limited extent.

Up to this time the courts have dealt with the constitutionality of marginal land condemnations under limited conditions presented by specific cases rather than on the basis of the broad principles involved.

The increment tax and special assessments are other methods by which governmental agencies may recover part of the increased values resulting from a street or road improvement. The former has not been used extensively in the United States. Although special assessments are not employed as extensively now as they were 10 years ago, they are still being used in financing many street improvements.

REGISTRATION OF ALIENS BEING MADE

As part of the National Defense program, a Nationwide registration of aliens is being conducted from August 27 through December 26, 1940, by the Immigration and Naturalization Service of the Department of Justice. Registration will take place in the post offices of the nation.

Registration is made compulsory by a specific act of Congress, the Alien Registration Act of 1940, which requires all noncitizens to register during the 4-month official registration period. The law requires that all aliens 14 years or older are to be registered and fingerprinted. Alien children under 14 years of age will be registered by their parents or guardians. When alien children reach their fourteenth birthday, they will be required to register in person and be fingerprinted.

A fine of $1,000 and imprisonment of 6 months is prescribed by the Alien Registration Act for failure to register, for refusal to be fingerprinted, or for making registration statements known to be false.

As part of its educational program to acquaint noncitizens with the registration requirements, the Alien Registration Division has distributed more than 5 million specimen forms listing the questions asked of aliens at registration time. Besides the usual questions for establishing identification, the questionnaire asks the alien to tell how and when he entered the country, the method of transportation he used to get here, the name of the vessel on which he arrived.

To make their registration easier, aliens are being asked to fill out sample forms and take them to post offices where they will be registered and fingerprinted. Every registered alien will receive by mail a receipt card which serves as evidence of his registration. Following registration, the act requires all aliens, as well as parents or guardians of alien children, to report changes of residence address within five days of the change.

The Alien Registration Act was passed so that the United States Government may determine exactly how many aliens there are, who they are, and where they are. The act provides that all records be kept secret and confidential. They will be available only to persons approved by the Attorney General of the United States.

Fingerprinting of aliens carries no stigma whatsoever. Members of the United States Army and Navy are all fingerprinted, as are many Government workers. Because fingerprinting is the only infallible method of accurate identification, the United States Government has adopted it as part of its registration program.

The Immigration and Naturalization Service asks for the cooperation of all citizens in carrying out the Alien Registration program in a friendly manner so that our large foreign population is not antagonized. It is suggested that citizens may be of great help to their noncitizen neighbors or relatives by explaining to those who do not speak English well what the registration is, where aliens go to register, and what information they must give.

HIGHWAY RESEARCH BOARD WILL MEET IN DECEMBER

The Twentieth Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., Wednesday through Friday, December 4–6, 1940.

Reports on highway research investigations will be presented, and the formal meetings of the Board will be supplemented with open meetings for informal discussion.

Meetings of the various committees, many of which will be open to the public, will be held December 2 and 3. A program of reports will be announced by the Board about November 1.

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STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JULY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 645,709	$ 320,567	29.7	$ 5,225,276	$ 2,598,023	171.1	$ 1,400,750	$ 697,620	43.4	$ 2,650,005
Arizona				1,344,580	856,718	59.9	206,933	95,570	4.5	1,640,391
Arkansas				2,695,225	1,364,064	132.4	619,509	303,887	18.1	1,369,899
California	952,021	501,730	17.0	7,359,448	3,808,796	111.8	3,750,586	2,020,970	76.7	2,432,345
Colorado	524,219	295,450	16.6	2,191,348	1,215,514	96.2	694,258	391,283	101.5	2,456,023
Connecticut	15,000	7,500		2,327,350	1,139,385	20.4	344,063	164,829	1.4	1,193,805
Delaware	52,661	26,330	.9	1,224,994	612,174	10.4	900,394	423,082	21.3	1,048,125
Florida	152,500	76,250	1.1	3,811,684	1,895,563	115.8	786,995	393,498	17.1	2,340,569
Georgia	308,788	154,394	26.5	6,161,566	3,080,783	299.3	4,584,579	2,292,790	151.1	5,120,822
Idaho	302,674	185,128	14.1	1,179,094	712,603	102.3	392,091	208,464	58.5	1,798,492
Illinois	386,000	193,000	7.2	8,699,498	4,349,584	205.1	1,881,200	939,870	25.2	3,771,293
Indiana	1,388,510	694,255	28.8	6,668,375	3,327,729	134.3	1,825,160	912,500	23.9	1,888,265
Iowa	568,768	214,690	4.0	4,731,594	2,163,912	164.1	3,774,266	1,772,760	120.9	633,771
Kansas	530,462	265,231	26.2	6,235,304	3,133,637	374.6	2,735,894	1,351,442	188.2	3,973,422
Kentucky	1,291	645		3,888,620	1,942,754	76.8	1,424,795	712,398	65.9	3,157,043
Louisiana				11,874,204	2,980,476	37.3	2,536,473	1,256,640	60.0	2,920,124
Maine	258,080	129,040	4.4	1,201,397	600,698	30.3	420,231	210,115	10.6	694,737
Maryland	68,000	34,000	.9	3,260,973	1,552,169	41.9	1,108,560	541,180	11.0	1,326,943
Massachusetts	225,870	112,854	1.2	1,928,380	960,388	19.4	2,293,058	1,140,914	15.4	2,803,104
Michigan	337,502	168,751	8.1	9,394,090	4,608,944	291.0	2,624,300	1,312,150	64.0	1,192,007
Minnesota	330,562	164,900	26.8	6,331,438	3,148,247	439.4	3,099,210	1,544,560	198.6	3,430,639
Mississippi	3,100	1,550		6,373,878	2,791,545	318.7	1,905,860	846,480	88.0	2,209,982
Missouri	1,151,071	575,536	49.4	5,744,808	2,859,986	200.5	3,754,172	1,410,657	88.0	4,670,548
Montana	802,893	455,365	48.6	3,596,776	2,034,753	241.2	606,783	332,190	31.6	4,024,385
Nebraska	338,938	169,469	36.3	6,059,689	2,926,296	704.9	1,774,555	887,278	221.3	2,823,865
Nevada	442,379	379,766	19.5	1,735,552	1,502,666	84.8	24,432	21,298	5.0	1,084,270
New Hampshire	84,632	41,167	1.2	1,503,338	740,727	38.2	275,114	132,008	5.9	877,857
New Jersey	1,089,800	544,670	8.9	4,416,148	2,208,074	32.9	130,840	65,420		2,167,494
New Mexico	29,397	17,720	9.8	1,814,003	1,119,004	109.0	507,069	305,661	23.4	1,651,760
New York	1,089,560	540,992	18.4	14,362,481	7,018,345	223.5	3,296,177	1,474,939	41.7	2,585,309
North Carolina	865,960	432,050	32.0	5,131,911	2,566,517	262.4	1,055,889	527,120	38.5	2,236,929
North Dakota	211,510	113,362	14.3	3,019,698	1,691,219	248.0	3,575,584	1,840,914	307.8	3,247,421
Ohio	177,860	88,930	1.5	11,820,175	5,885,758	120.6	4,378,960	2,189,480	32.6	5,249,449
Oklahoma	304,412	161,341	19.3	3,108,152	1,645,252	86.4	2,158,787	1,090,682	102.7	4,171,640
Oregon	339,590	204,920	15.2	3,630,392	2,173,985	142.3	419,674	245,900	11.4	1,342,647
Pennsylvania	359,582	177,451	3.9	11,211,266	5,555,494	127.5	3,346,276	1,653,902	24.6	3,559,276
Rhode Island	115,572	57,780	1.2	1,214,836	606,315	12.1	418,507	208,955	4.3	998,213
South Carolina	22,600	11,300	.1	2,511,504	1,210,143	156.5	911,692	440,475	114.8	2,396,782
South Dakota	479,320	264,740	62.0	4,028,360	2,356,230	510.3	1,520,780	875,760	299.8	3,145,532
Tennessee	83,630	41,815	.6	3,669,120	1,834,560	101.4	1,341,514	670,757	61.2	4,018,539
Texas	788,300	393,460	71.9	8,323,649	4,113,302	425.5	2,530,291	1,248,452	104.5	7,636,052
Utah	194,920	141,845	15.0	1,175,850	871,753	71.7	263,010	146,540	5.8	1,062,121
Vermont				1,415,695	707,247	41.4	595,687	297,357	15.5	275,025
Virginia	33,900	16,950	1.7	3,316,645	1,597,849	79.5	412,903	174,852	13.7	2,268,502
Washington	277,742	145,094	2.6	4,062,519	2,139,435	86.3	520,407	275,200	11.3	1,067,931
West Virginia	199,250	98,445	7.6	3,177,150	1,582,286	93.1	1,259,990	629,145	33.0	1,703,221
Wisconsin	671,745	321,400	39.8	5,116,484	2,525,210	142.0	1,154,922	554,580	76.1	3,577,236
Wyoming	368,478	232,616	39.8	1,298,978	827,943	128.1	423,445	260,392	70.0	1,119,559
District of Columbia				456,324	192,050	4.1	189,200	94,600	1.4	424,696
Hawaii	29,184	11,010		217,910	112,602	2.8	682,741	370,516	11.6	1,486,733
Puerto Rico				1,389,245	686,970	25.6	228,708	113,720	.6	770,080
TOTALS	17,603,942	9,185,459	734.1	222,636,974	110,135,677	7,755.1	77,067,274	38,040,752	3,123.4	121,694,878

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JULY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 96,617	$ 48,100	1.3	$ 1,038,097	$ 517,420	56.4	$111,900	$ 45,000	0.3	$ 490,992
Arizona	53,782	17,866	4.7	172,207	124,309	6.6	80,472	42,220	6.2	273,580
Arkansas	26,741	13,365	3.9	215,098	117,494	18.0	133,901	66,923	11.8	219,590
California	58,176	34,039	4.8	716,211	386,118	27.9	288,114	156,151	9.3	841,383
Colorado				152,679	63,187	1.2	54,731	30,846	.7	196,189
Connecticut				367,226	178,009	4.6	33,907	16,890	.2	189,178
Delaware				128,160	56,367	12.7	40,371	19,751	2.9	268,125
Florida				29,916	14,958	.6	795,359	393,888	14.3	214,839
Georgia	25,161	12,581	4.6	559,299	264,649	53.1	421,337	210,668	32.2	1,117,569
Idaho				36,379	22,413	6.0	58,168	34,811	10.6	240,904
Illinois	230,000	115,000	16.0	1,812,985	889,318	62.6	413,400	206,700	20.4	317,965
Indiana	114,100	57,050	7.5	357,470	177,531	23.5	99,006	49,400	6.2	948,563
Iowa	497,225	236,275	85.9	1,152,202	548,566	216.6	948,751	448,259	251.5	307,368
Kansas	154,352	77,176	1.6	430,515	218,166	27.9	574,953	287,476	51.2	1,248,215
Kentucky	46,276	20,000	2.1	714,930	252,030	58.6	364,406	96,800	19.1	351,278
Louisiana				297,930	148,910	25.7				455,182
Maine	17,600	8,011	1.2	166,184	79,783	9.1	145,320	70,560	7.5	10,017
Maryland				131,996	65,998	5.5	46,000	23,000	2.7	427,378
Massachusetts	110,536	54,886	2.7	512,049	253,775	10.5	30,530	15,265	.2	487,958
Michigan	19,560	9,780	2.4	1,313,219	659,999	103.6	453,260	226,630	35.5	560,988
Minnesota	6,827	3,414	1.4	659,812	329,190	77.9	321,036	160,518	58.6	1,104,901
Mississippi				657,362	323,181	22.6	304,900	140,915	18.2	573,008
Missouri	116,914	58,457	16.3	544,816	268,997	55.4	283,956	114,429	45.5	719,677
Montana	28,040	15,904	4.6	444,673	251,422	58.3	230,748	130,211	19.9	658,675
Nebraska	40,042	19,862	6.7	724,548	362,274	100.4	251,716	120,761	39.6	288,859
Nevada	44,434	38,217	.9	110,220	95,265	32.7	105,984	89,156	9.0	104,172
New Hampshire	48,387	23,241	2.2	37,254	16,906		71,651	35,453	1.2	162,156
New Jersey				583,590	291,715	19.6				528,956
New Mexico	101,564	63,386	13.1	266,287	166,062	13.8	150,797	72,336	.9	190,111
New York	257,480	128,740	6.1	2,176,855	1,053,350	67.9	887,700	348,898	21.6	178,256
North Carolina	43,370	21,685	6.3	1,003,503	502,718	86.5	47,000	23,300	4.9	330,897
North Dakota				184,584	100,535	1.4	27,520	14,750	2.4	1,014,638
Ohio	56,294	28,081	3.2	2,517,762	1,265,656	89.9	724,661	360,960	14.5	831,375
Oklahoma	67,377	35,754	6.0	608,510	322,621	41.8	267,180	116,876	12.4	924,501
Oregon	35,193	21,240	4.2	304,745	160,260	45.5	255,698	108,960	21.7	256,964
Pennsylvania	369,736	178,763	16.0	1,572,832	786,416	45.2	370,751	184,467	8.8	243,018
Rhode Island				230,054	115,001	3.6				95,049
South Carolina				651,540	250,827	61.0	184,830	74,850	56.4	228,284
South Dakota				3,624	3,624					1,280,913
Tennessee	43,036	21,518	3.1	101,256	50,628	7.7				1,007,123
Texas	483,976	238,950	60.6	912,635	440,413	122.7	220,990	95,030	24.2	1,062,153
Utah				6,700	5,000		210,427	128,600	22.3	163,707
Vermont	98,200	30,800	4.1	372,691	108,755	13.5	42,925	12,500	1.8	22,500
Virginia	57,340	24,667	1.5	525,974	244,960	31.9	51,326	22,779	.7	328,772
Washington	90,379	47,400	2.8	297,603	156,539	21.9	280,829	149,600	9.5	240,072
West Virginia	39,400	19,700	1.6	267,219	133,084	15.2	79,808	39,904	4.4	448,189
Wisconsin	130,102	64,750	.4	510,901	255,370	16.4	287,506	143,300	13.0	653,467
Wyoming	102,039	64,414	19.5	287,013	168,301	19.5	114,120	72,944	7.9	113,561
District of Columbia				41,492	20,246	.6	74,900	37,450	.7	29,296
Hawaii				278,748	139,878	8.7				158,778
Puerto Rico				302,224	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	3,710,256	1,853,072	319.3	27,491,779	13,575,834	1,826.3	10,998,033	5,267,360	905.0	23,189,747

U. S. GOVERNMENT PRINTING OFFICE: 1940

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

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ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

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Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

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MISCELLANEOUS PUBLICATIONS

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The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JULY 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama						$ 770,410	$ 770,327	6		1	$ 17,000	$ 17,000			5	$ 874,810
Arizona	$ 159,652	$ 134,732	1			201,045	193,841	3			2,362	2,362			1	264,581
Arkansas	10,230	10,230			4	1,154,833	1,150,686	8	1		188,872	188,659	4		9	454,866
California	4,090	4,090			1	1,057,831	879,350	5	1		18,434	18,434			9	1,687,509
Colorado						277,100	277,100	1			3,401	3,401			1	922,774
Connecticut	153,792	148,343		1		631,483	625,932	7								451,839
Delaware						146,647	146,647	1		11	38,943	38,943			6	479,112
Florida	111,728	107,229	1			91,620	91,620	1		1	164,314	163,984	2		16	1,291,952
Georgia	63,610	63,610	1		5	762,136	762,136	10	4	1	471,114	471,114	4	2	11	1,964,438
Idaho						207,740	204,310	5			79,646	79,646			31	430,468
Illinois	109,232	107,622	1		4	2,553,797	2,311,265	10	2	19	485,572	420,829	1		88	1,938,341
Indiana	22,784	22,784			11	810,699	810,699	5	2	38	272,614	272,614	1		36	1,109,668
Iowa	57,614	53,950			16	280,798	229,488	3		23	306,127	269,234	3		56	1,208,815
Kansas	4,402	4,402			2	879,093	878,615	12		3	254,202	254,202	3	2	11	1,115,081
Kentucky	112,479	112,479	2		4	1,046,551	1,046,551	11		1	46,932	46,932	2		6	574,789
Louisiana						440,618	387,123	3			627,884	570,153	11			789,393
Maine	69,077	69,077		1	2	81,388	81,388	1		1	153,940	153,940	1	1	3	250,472
Maryland						634,509	602,716	3	2		6,400	6,400			2	798,625
Massachusetts	15,710	15,710				336,591	326,168	1								2,025,042
Michigan	95,224	95,224	1	1	5	1,814,350	1,814,350	8	3	5	281,816	281,816		1	15	973,791
Minnesota	396,615	396,615	3	1		1,522,016	1,520,923	13	3		239,678	239,678		2	13	1,076,573
Mississippi						481,419	481,419	5	1		49,200	49,200	3		1	895,624
Missouri						1,183,905	1,183,905	6	4		1,269,305	813,885	3	1	1	1,508,162
Montana						271,074	201,433	2		1	171,535	171,535	3		3	436,252
Nebraska						734,112	734,112	6	2	7	107,678	107,678	1		3	664,916
Nevada						45,546	45,546	1			112,146	112,146	1		25	113,864
New Hampshire						181,248	181,212	4								393,572
New Jersey						588,109	588,109	3			225,737	225,737	2			1,242,438
New Mexico	10,377	10,377			3	232,961	232,961	2		1	7,609	7,609			2	646,594
New York	307,110	306,000	2	1		3,210,539	3,150,247	10	14		820,772	701,336		5	8	2,997,444
North Carolina	92,275	92,275	1	1	3	936,458	936,458	12	2	9	125,520	125,520	1	1	16	898,855
North Dakota	306,540	306,540	4		1	204,040	204,040	3		9	41,110	41,110			12	771,928
Ohio						2,784,713	2,713,761	14	3	6	41,990	41,990			7	3,026,369
Oklahoma	47,751	47,625	1		1	506,902	506,002	9	1	1	478,881	473,495	3		57	1,898,553
Oregon						217,041	125,939	3			245,806	189,579	1			363,725
Pennsylvania	34,709	34,709		1		1,765,493	1,756,017	15	1		709,817	709,817	6			4,388,349
Rhode Island						194,789	194,789		1							95,913
South Carolina	56,842	56,842	2		1	379,543	379,543	3	2	8	80,943	80,540		1	14	1,070,979
South Dakota						173,312	177,452	3	1	3	286,830	270,880	7		1	1,092,587
Tennessee						244,396	244,396	2	2		124,741	124,741		1		1,758,339
Texas	133,099	132,800	1			1,503,148	1,430,273	13		4	727,820	662,130	7	1	6	2,134,382
Utah						50,171	50,171			13	19,287	19,287			7	328,820
Vermont						209,371	209,371		2	1	15,144	15,144			5	245,989
Virginia	9,500	9,500			3	287,968	286,302	4	2		24,182	24,182			6	1,187,095
Washington	70,354	70,354		1	2	285,753	284,253	4	1		81,575	81,575	1		7	513,498
West Virginia						111,510	111,510	1			122,400	122,400	1		2	1,136,132
Wisconsin	199,073	198,812	3		2	1,074,731	1,030,870	6	4	8	11,201	11,201			3	1,373,756
Wyoming						540,923	540,923	5								192,955
District of Columbia						8,868	8,868				48,000	48,000	1			152,203
Hawaii						194,036	194,028	2			9,494	9,494			2	289,822
Puerto Rico						584,007	579,336	11								414,857
TOTALS	2,653,869	2,611,931	24	8	70	34,892,341	33,879,481	266	61	175	9,617,974	8,739,552	73	18	507	52,916,911

PUBLIC ROADS

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FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

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PUBLIC ROADS ADMINISTRATION
D. M. BEACH, *Editor*
Volume 21, No. 7 September 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

DESIGNING CONCRETE MIXTURES FOR PAVEMENTS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **W. F. KELLERMANN**, Materials Engineer

THE PURPOSE of this report is to describe a method of investigating the flexural strength of concrete in connection with the problem of designing concrete mixtures for pavements and to present the results of a series of laboratory tests which demonstrate how flexural strength may vary over a wide range due to the characteristics of the aggregates employed.

All pavement concrete, particularly that laid in the Northern States, must be designed so as to afford maximum resistance to weathering agencies. Assuming that the constituent materials are durable, it is generally agreed that this may be accomplished either by placing a maximum limit on the water-cement ratio or by requiring a cement content sufficiently high to insure that the maximum allowable water content will not be exceeded. The necessity for limiting the water-cement ratio to insure durability applies to all concrete exposed to the weather.

Insofar as strength characteristics are concerned, concrete for most purposes need only be investigated for compressive strength. However, compressive strength is not of primary importance in concrete for pavement slabs because of the character of the stresses to which such slabs are subjected. Live loads and changes in temperature and moisture, either alone or in combination, produce tensile and flexural stresses which pavements must resist in order to perform the function for which they are designed. Of the two, the flexural stresses are the more important. For this reason, flexural or bending stresses rather than compressive stresses become critical in cases where the concrete mixture is to be designed for use in highway pavements. Therefore the designer of concrete paving mixtures must give consideration not only to the factors that affect durability but also to those variables that affect flexural strength.

TESTS MADE TO DETERMINE CEMENT FACTOR FOR VARIOUS COMBINATIONS OF AGGREGATES

The specifications for pavement concrete of the American Association of State Highway Officials specify that the proportions shall be based on laboratory tests and shall be such that, in the judgment of the engineer, they will assure durable concrete of the plasticity and workability required, and which will attain at the age of 14 days a modulus of rupture not less than 550 pounds per square inch when tested by the third point method of loading. In order to assure durability it is further specified that the net water-cement ratio shall in no case exceed 0.80 by volume (6.0 gallons per sack of cement).

The tests reported in this paper were made in an investigation of the design of concrete mixtures in which 25 different combinations of fine and coarse aggregate were used, the requirement being compliance with the above specifications. The work was done in the laboratory of the Public Roads Administration during 1936 at the request of the State Highway and Public Works Commission of North Carolina. The purpose was to establish the cement factor required for various combinations of available aggregates, the information thus obtained to be used as the basis for bidding. Seven sands and 13 coarse aggregates, all commercially available in North Carolina, were investigated. The 25 combinations of materials were selected on the basis of economic availability and represented practically all combinations of aggregates that were likely to be encountered in practice in that State.

As will be noted from table 1 the sands varied in grading over a wide range, the fineness modulus of the finest being 2.12 and that of the coarsest, 3.37. All coarse aggregates were separated into three sizes and recombined for test in accordance with the grading shown in table 2, the maximum size being 2 inches. This table also gives the mineral composition of both the fine and coarse aggregates as well as their physical properties. One lot of cement, meeting all A. S. T. M. requirements, was used throughout.

In order to determine the required cement content for each combination of materials it was decided to establish directly the relation between cement content and flexural strength at 14 days, using mixtures with five different cement factors as follows: 4.4, 5.2, 6.0, 6.8, and 7.2 sacks of cement per cubic yard of concrete. This procedure also afforded an opportunity to establish the corresponding relations between water-cement ratio and flexural strength.

TABLE 1.—*Sieve analysis of fine aggregates for concrete mixes using North Carolina aggregates*

Fine aggregate	Percentage retained on sieve no.—						Fineness modulus
	4	8	16	30	50	100	
1	0	0	2.8	35.8	77.3	95.9	2.12
2	0	0	5.3	42.8	79.4	94.4	2.22
3	.5	4.0	19.6	58.0	93.1	99.4	2.75
4	.2	4.1	21.2	63.2	93.2	99.0	2.81
5	0	7.3	26.8	62.1	89.3	97.2	2.83
6	.1	7.1	30.4	68.3	91.0	98.4	2.95
7	.3	14.6	44.2	81.2	97.6	99.4	3.37

The decision to design the mixes on the basis of a fixed cement factor rather than by the use of fixed water-cement ratios was made because of the fact that in the North Carolina specifications the final proportions are stated in terms of a fixed cement factor for each aggregate combination. The problem, therefore, resolved itself into one of designing 125 different concrete mixes: Five cement contents with each of the 25 combinations of aggregates. The problem was complicated by the fact that both angular and rounded coarse aggregates were used in combination with sands graded from extremely fine to extremely coarse.

In keeping with North Carolina practice the different mixes were designed with a view to maintaining a minimum of sand consistent with satisfactory workability at a consistency corresponding to a slump of 2½ inches. This was accomplished by making numerous trial batches, the ratio of fine to coarse aggregate

Table 2.—*Physical properties of aggregates for concrete mixes using North Carolina aggregates*

Aggregate	Type	Bulk specific gravity	Weight per cu. ft.[1]		Voids		Absorption	Abrasion loss	
			Dry rodded	Dry loose	Dry rodded	Dry loose		Los Angeles[2]	Deval[3]
STONE									
			Pounds	*Pounds*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	Dolomite	2.85	105	97	41	45	0.27	30.4	4.3
2	do	2.82	103	93	42	47	.56	33.0	5.0
3	Limestone	2.79	102	92	41	47	.44	21.1	2.6
4	Granite	2.69	101	91	40	46	.46	51.1	4.0
5	do	2.65	102	93	38	44	.46	53.5	2.6
6	do	2.64	102	92	38	44	.58	37.9	2.4
7	do	2.63	100	91	39	44	.52	58.4	3.9
8	do	2.62	95	87	42	47	.28	30.1	2.0
9	do	2.62	100	90	39	45	.44	41.9	3.6
GRAVEL									
10	Gneiss	2.67	106	98	36	41	1.19	47.9	18.5
11	Quartz	2.63	111	103	32	37	.22	53.7	14.6
12	do	2.63	109	103	34	37	.28	40.9	8.2
13	do	2.63	111	104	32	37	.32	54.9	19.4
SAND									
1	Quartz	2.64	98	91	40	45	0.40		
2	do	2.63	100	93	39	43	.50		
3	do	2.63	98	93	40	43	.40		
4	do	2.64	98	92	40	44	.40		
5	do	2.66	103	97	38	42	.35		
6	do	2.64	101	94	39	43	.32		
7	do	2.60	99	94	39	42	.70		

[1] All coarse aggregates proportioned to give following grading in concrete. Unit weights determined for this grading:

	Percent
Total retained on 2-inch sieve	0
Total retained on 1½-inch sieve	15
Total retained on ¾-inch sieve	70
Total retained on No. 4 sieve	100
Fineness modulus	7.75

[2] Grading A used with both stone and gravel.

[3] Grading A used with gravel. Stone sample consisted of 50 pieces weighing 5 kilograms.

being adjusted, in each case, until, in the opinion of the operator, the minimum sand content was reached.

The final proportions for each of the 5 cement factors and for each of the 25 aggregate combinations are shown in table 3. This table includes, in addition to the mix proportions by weight, the water-cement ratio by volume, the value of W_c, that is, the volume of water in a unit volume of concrete, the ratio b/b_0 as defined by Talbot and Richart,[1] the mortar voids ratio,[2] the percentage of sand by weight, the fineness modulus of the combined aggregate, and the resulting slump in inches.

SEVERAL THEORIES OF MIX DESIGN TRIED

In view of the fact that it was necessary to design 25 different mixes for each cement content, attempts were made to apply certain theories of mix design to the problem of determining the proper percentage of sand to use in each case. An attempt to use the fineness modulus theory of Abrams[3] proved unsuccessful due to the fact that a fixed value for maximum permissible fineness modulus could not be used, even in the case of a given sand combined with several coarse aggregates of the same general particle shape.

Next, an attempt was made to design on the basis of a fixed value for the mortar voids ratio, that is, a constant excess of mortar over the amount necessary to fill the voids in the coarse aggregate. This was found satisfactory so long as particle shape remained reasonably constant. However, as will be noted in table 3, a

Table 3.—*Data on concrete mixes using North Carolina aggregates*

CEMENT FACTOR—4.4 SACKS PER CUBIC YARD

Aggregate		Proportions by weight	Water-cement ratio by volume	W_c[1]	b/b_0	Mortar voids ratio	Sand to total aggregate by weight	Fineness modulus of combined aggregate	Slump
Fine	Coarse								
		Pounds					*Percent*		*Inches*
1	1	94:254:548	1.04	0.169	0.920	1.19	31.7	5.88	2.4
	2	94:264:524	1.08	.176	.920	1.18	33.5	5.79	2.3
	3	94:267:519	1.06	.173	.920	1.18	34.0	5.77	2.3
	10	94:226:553	.97	.158	.920	1.21	29.0	6.11	2.5
2	4	94:256:514	1.04	.169	.920	1.19	33.2	5.88	2.5
	5	94:240:526	1.02	.166	.920	1.20	31.3	6.01	2.7
	6	94:245:520	1.01	.164	.920	1.20	32.0	5.98	2.5
	7	94:246:515	1.02	.166	.920	1.20	32.3	5.96	2.4
	8	94:263:492	1.05	.171	.920	1.19	34.8	5.83	2.6
	9	94:251:508	1.02	.166	.920	1.19	33.1	5.93	2.4
	11	94:205:583	.86	.140	.920	1.23	26.0	6.31	2.6
	13	94:200:587	.86	.140	.920	1.24	25.4	6.35	2.5
3	4	94:272:492	1.07	.174	.880	1.30	35.6	5.94	2.4
	5	94:263:501	1.03	.168	.880	1.31	34.4	6.02	2.6
	6	94:263:497	1.04	.169	.880	1.31	34.6	6.02	2.5
	7	94:267:492	1.03	.168	.880	1.31	35.2	5.99	2.4
	9	94:272:486	1.03	.168	.880	1.30	35.9	5.96	2.5
4	13	94:224:550	.95	.155	.860	1.45	28.9	6.32	2.6
5	7	94:274:492	1.01	.164	.880	1.31	35.8	6.00	2.5
	9	94:276:486	1.03	.168	.880	1.30	36.2	5.98	2.5
6	4	94:280:480	1.10	.179	.860	1.36	36.8	5.96	2.4
	6	94:277:486	1.03	.168	.860	1.37	36.3	6.01	2.4
	11	94:237:545	.90	.147	.860	1.44	30.3	6.30	2.6
7	7	94:297:458	1.04	.169	.820	1.50	39.3	6.02	2.3
	12	94:261:519	.89	.145	.820	1.59	33.5	6.27	2.4

[1] Volume of water per unit volume of concrete.

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[1] A. N. Talbot and F. E. Richart, University of Illinois Engineering Experiment Station Bulletin 137. The term b/b_0 is defined as the ratio of the absolute volume of coarse aggregate in a unit volume of concrete (b) to the absolute volume of coarse aggregate in a unit volume of coarse aggregate (b_0). Stated in different terms, it is the apparent volume of coarse aggregate in a unit volume of concrete. The values given in table 3 are on a dry-loose basis.

[2] The ratio of the volume of mortar in a unit volume of concrete to the volume of the voids in the coarse aggregate, determined in a dry-loose condition.

[3] The Design of Concrete Mixtures, by D. A. Abrams. Bulletin No. 1, Structural Materials Research Laboratory, Lewis Institute.

TABLE 3.—*Data on concrete mixes using North Carolina aggregates*—Continued

CEMENT FACTOR—5.2 SACKS PER CUBIC YARD

Aggregate		Proportions by weight	Water-cement ratio by volume	W_c [1]	b/b_0	Mortar voids ratio	Sand to total aggregate by weight	Fineness modulus of combined aggregate	Slump
Fine	Coarse								
		Pounds					*Percent*		*Inches*
1	1	94:208:459	0.87	0.168	0.910	1.22	31.2	5.90	2.5
	2	94:216:438	.91	.175	.910	1.21	33.0	5.81	2.6
	3	94:217:434	.90	.173	.910	1.21	33.3	5.80	2.5
	10	94:183:463	.82	.158	.910	1.24	28.3	6.15	2.6
2	4	94:207:430	.89	.171	.910	1.22	32.5	5.93	2.6
	5	94:199:438	.85	.164	.910	1.23	31.2	6.02	2.6
	6	94:199:435	.86	.166	.910	1.23	31.4	6.01	2.6
	7	94:202:430	.86	.166	.910	1.22	32.0	5.99	2.5
	8	94:215:410	.89	.171	.910	1.21	34.4	5.85	2.5
	9	94:205:425	.86	.166	.910	1.22	32.5	5.95	2.6
	11	94:169:486	.72	.139	.910	1.27	25.8	6.32	2.5
	13	94:162:491	.73	.141	.910	1.27	24.8	6.37	2.5
3	4	94:225:411	.89	.171	.870	1.32	35.4	5.96	2.5
	5	94:217:420	.85	.164	.870	1.34	34.1	6.04	2.6
	6	94:218:417	.85	.164	.870	1.34	34.3	6.02	2.6
	7	94:218:412	.87	.168	.870	1.33	34.6	6.02	2.5
	9	94:222:407	.87	.168	.870	1.33	35.3	5.99	2.6
4	13	94:184:459	.79	.152	.850	1.48	28.6	6.34	2.6
5	7	94:224:412	.85	.164	.870	1.33	35.2	6.03	2.6
	9	94:224:407	.87	.168	.870	1.33	35.5	6.02	2.6
6	4	94:231:401	.92	.177	.850	1.39	36.6	5.98	2.7
	6	94:224:407	.88	.170	.850	1.40	35.5	6.05	2.6
	11	94:194:455	.76	.146	.850	1.48	29.9	6.31	2.6
7	7	94:243:382	.88	.170	.810	1.53	38.9	6.03	2.5
	12	94:212:433	.76	.146	.810	1.63	32.9	6.30	2.5

CEMENT FACTOR—6.0 SACKS PER CUBIC YARD

Fine	Coarse	Proportions by weight	Water-cement ratio by volume	W_c [1]	b/b_0	Mortar voids ratio	Sand to total aggregate by weight	Fineness modulus of combined aggregate	Slump
1	1	94:173:393	0.76	0.169	0.900	1.24	30.6	5.94	2.5
	2	94:181:377	.78	.173	.900	1.24	32.4	5.84	2.6
	3	94:181:373	.78	.173	.900	1.24	32.7	5.84	2.7
	10	94:153:397	.71	.158	.900	1.27	27.8	6.17	2.6
2	4	94:174:369	.76	.169	.900	1.24	32.0	5.95	2.5
	5	94:162:377	.75	.167	.900	1.25	30.1	6.07	2.6
	6	94:166:372	.75	.167	.900	1.25	30.9	6.04	2.5
	7	94:167:369	.75	.167	.900	1.25	31.2	6.02	2.6
	8	94:181:352	.77	.171	.900	1.24	34.0	5.88	2.5
	9	94:167:365	.77	.171	.900	1.25	31.4	6.01	2.6
	11	94:138:417	.64	.142	.900	1.30	24.9	6.37	2.5
	13	94:133:422	.64	.142	.900	1.30	24.0	6.42	2.4
3	4	94:189:353	.77	.171	.860	1.36	34.9	5.98	2.7
	5	94:182:359	.74	.164	.860	1.37	33.6	6.06	2.7
	6	94:184:356	.74	.164	.860	1.37	34.1	6.04	2.6
	7	94:185:353	.74	.164	.860	1.37	34.4	6.02	2.4
	9	94:187:348	.75	.167	.860	1.36	35.0	6.00	2.7
4	13	94:155:394	.68	.151	.840	1.52	28.2	6.36	2.5
5	7	94:189:353	.73	.162	.860	1.37	34.9	6.04	2.6
	9	94:191:348	.74	.164	.860	1.36	35.4	6.02	2.6
6	4	94:196:344	.78	.173	.840	1.42	36.3	5.99	2.5
	6	94:191:348	.75	.167	.840	1.43	35.4	6.05	2.4
	11	94:165:389	.65	.144	.840	1.51	29.8	6.32	2.3
7	7	94:206:328	.75	.167	.800	1.56	38.6	6.05	2.4
	12	94:178:371	.66	.147	.800	1.67	32.4	6.32	2.5

CEMENT FACTOR—6.8 SACKS PER CUBIC YARD

Fine	Coarse	Proportions by weight	Water-cement ratio by volume	W_c [1]	b/b_0	Mortar voids ratio	Sand to total aggregate by weight	Fineness modulus of combined aggregate	Slump
		Pounds					*Percent*		*Inches*
1	1	94:150:343	0.65	0.164	0.890	1.28	30.4	5.95	2.3
	2	94:156:327	.68	.171	.890	1.26	32.3	5.85	2.6
	3	94:158:324	.67	.169	.890	1.26	32.8	5.84	2.6
	10	94:128:347	.63	.159	.890	1.30	26.9	6.21	2.6
2	4	94:148:321	.68	.171	.890	1.27	31.6	5.98	2.5
	5	94:139:327	.66	.166	.890	1.28	29.8	6.09	2.6
	6	94:143:324	.65	.164	.890	1.28	30.6	6.06	2.5
	7	94:141:322	.67	.169	.890	1.28	30.5	6.06	2.6
	8	94:153:307	.68	.171	.890	1.27	33.3	5.92	2.6
	9	94:144:317	.67	.169	.890	1.28	31.2	6.02	2.6
	11	94:115:364	.57	.144	.890	1.34	24.0	6.42	2.5
	13	94:110:368	.58	.146	.890	1.34	23.0	6.48	2.5
3	4	94:164:307	.66	.166	.850	1.39	34.8	5.98	2.5
	5	94:156:313	.65	.164	.850	1.41	33.3	6.08	2.6
	6	94:159:310	.64	.161	.850	1.40	33.9	6.05	2.5
	7	94:159:307	.65	.164	.850	1.40	34.1	6.04	2.6
	9	94:161:302	.66	.166	.850	1.39	34.8	6.02	2.6
4	13	94:130:343	.61	.154	.830	1.55	27.5	6.40	2.6
5	7	94:161:307	.65	.164	.850	1.40	34.4	6.07	2.6
	9	94:164:302	.65	.164	.850	1.39	35.2	6.03	2.7
6	4	94:165:301	.70	.176	.830	1.44	35.4	6.03	2.5
	6	94:161:303	.67	.169	.830	1.46	34.7	6.08	2.6
	11	94:137:340	.59	.149	.830	1.54	28.7	6.38	2.5
7	7	94:178:286	.65	.164	.790	1.59	38.4	6.05	2.5
	12	94:152:323	.58	.146	.790	1.71	32.0	6.34	2.4

TABLE 3.—*Data on concrete mixes using North Carolina aggregates*—Continued

CEMENT FACTOR—6.8 SACKS PER CUBIC YARD—Continued

CEMENT FACTOR—7.2 SACKS PER CUBIC YARD

Fine	Coarse	Proportions by weight	Water-cement ratio by volume	W_c [1]	b/b_0	Mortar voids ratio	Sand to total aggregate by weight	Fineness modulus of combined aggregate	Slump
1	1	94:137:322	0.63	0.168	0.885	1.28	29.8	5.93	2.6
	2	94:143:308	.65	.173	.885	1.27	31.7	5.88	2.6
	3	94:145:305	.61	.171	.885	1.27	32.2	5.86	2.5
	10	94:119:325	.60	.160	.885	1.31	26.8	6.23	2.6
2	4	94:136:302	.64	.171	.885	1.28	31.1	6.00	2.5
	5	94:128:309	.62	.165	.885	1.30	29.3	6.12	2.5
	6	94:130:306	.62	.165	.885	1.29	29.8	6.10	2.5
	7	94:133:302	.62	.165	.885	1.29	30.6	6.06	2.4
	8	94:141:289	.64	.171	.885	1.28	32.8	5.94	2.4
	9	94:135:299	.62	.165	.885	1.29	31.1	6.04	2.4
	11	94:105:311	.55	.147	.885	1.35	23.5	6.45	2.5
	13	94:102:345	.55	.147	.885	1.35	22.8	6.49	2.4
3	4	94:151:289	.63	.168	.845	1.40	34.3	6.01	2.6
	5	94:144:294	.61	.163	.845	1.42	32.9	6.10	2.6
	6	94:146:292	.61	.163	.845	1.41	33.3	6.08	2.6
	7	94:146:289	.62	.165	.845	1.41	33.6	6.07	2.6
	9	94:149:284	.62	.165	.845	1.41	34.4	6.04	2.6
4	13	94:120:322	.58	.155	.825	1.58	27.1	6.41	2.5
5	7	94:149:289	.61	.163	.845	1.41	34.0	6.09	2.6
	9	94:151:285	.62	.165	.845	1.41	34.6	6.06	2.5
6	4	94:155:280	.66	.176	.825	1.46	35.6	6.02	2.5
	6	94:150:285	.63	.168	.825	1.49	34.5	6.09	2.4
	11	94:127:318	.56	.119	.825	1.57	28.5	6.39	2.5
7	7	94:165:267	.62	.165	.785	1.62	38.2	6.06	2.5
	12	94:141:301	.55	.147	.785	1.74	31.7	6.35	2.5

change from angular to rounded coarse aggregate resulted in an increase in the value of this ratio. (Compare combinations 2–4 to 2–9, inclusive, with combinations 2–11 and 2–13.)

This particular difficulty was overcome by using a fixed value of b/b_0 in place of a constant mortar voids ratio. After making many trial batches it was found that, for a given cement content and a given sand, a constant value of b/b_0 could be used irrespective of type of coarse aggregate. (See table 3.) It was noted, furthermore, that certain consistent changes in b/b_0 resulted from changing the cement factor and sand grading. Thus, for a given sand, table 3 shows that an increase in the cement factor of 0.8 sack resulted in a decrease of 0.01 in b/b_0 for equal workability. Furthermore, as the sand used became coarser, it was found necessary to reduce the value of b/b_0. Thus at 6.0 sacks per cubic yard the value of this ratio decreased from 0.90 for sand No. 1 (F. M. = 2.12) to 0.80 for sand No. 7 (F. M. = 3.37).

It is believed that the systematic variation in b/b_0 with changes in cement content revealed by these tests together with the principle stated by Lyse[4] in 1932 to the effect that, for a given combination of materials and a given consistency, the total quantity of water per unit of volume of concrete (W_c) is constant regardless of the cement content, makes it possible to simplify considerably the problem of designing mixtures of varying cement content. Having established by trial the proper value of b/b_0 to use with given aggregates and with a given cement factor, the proportions required for the same consistency with any other cement factor may be obtained by computation, provided the aggregates have the same gradations as those used in the trial batch. This will be illustrated by an example.

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[4] Inge Lyse, Tests of Consistency and Strength of Concrete Having Constant Water Content. Proceedings A. S. T. M., vol. 32, pt. II, 1932.

Given a mix having:

The proportions 94:274:492 by weight.
Water-cement ratio 1.01 by volume.
Specific gravities of materials: Cement, 3.16; fine aggregate, 2.66; coarse aggregate, 2.63.

Percentage of voids in coarse aggregate (dry–loose), 44.5, the corresponding solid volumes per 1-sack batch would be as follows:

		Cu. ft.
Cement	$\frac{94}{62.4\times3.16}$	$=0.48$
Fine aggregate	$\frac{274}{62.4\times2.66}$	$=1.65$
Coarse aggregate	$\frac{492}{62.4\times2.63}$	$=3.00$
Water (W/C)		$=1.01$
Yield		$=6.14$

Cement factor $=\frac{27}{6.14}=4.4$ sacks per cubic yard.

Bulk volume of coarse aggregate $=\frac{3.00}{1.00-0.445}$ $=5.41$ cubic feet.

Then $b/b_0=\frac{5.41}{6.14}=0.88$.

Water per unit volume of concrete (W_c) $=\frac{1.01}{6.14}=0.164$.

Having analyzed the above mix and determined the values of b/b_0 and W_c as 0.88 and 0.164, respectively, another mix will be designed with the same materials but with a cement factor of 6.8 sacks of cement per cubic yard of concrete, an increase of 2.4 sacks per cubic yard.

Following the procedure outlined, the value for W_c for the new mix would remain constant at 0.164. The new value for b/b_0 is determined as follows: For each increase of 0.8 sack per cubic yard in cement factor, b/b_0 is decreased 0.01. Therefore, the new value of b/b_0 is $0.88-\frac{2.4}{0.8}\times0.01=0.85$.

The yield per 1-sack batch for the new mix would be $\frac{27}{6.8}=3.97$ cubic feet and the apparent or bulk volume of coarse aggregate would be $3.97\times0.85=3.37$ cubic feet. The corresponding solid volume of coarse aggregate $(b)=3.37\times(1-0.445)=1.87$ cubic feet; the solid volume of cement $(c)=0.48$ cubic feet; and the volume of water $(W/C)=3.97\times0.164=0.65$ cubic feet; making a total of 3.00 cubic feet. The only unknown quantity remaining is the volume of fine aggregate, which is determined by subtracting the sum of the solid volumes of coarse aggregate, cement, and water from the total yield. Therefore the solid volume of fine aggregate $=3.97-3.00=0.97$ cubic feet. The complete proportions per 1-sack batch for the new mix would be as follows:

	Cu. ft.
Cement (solid)	0.48
Fine aggregate (solid)	0.97
Coarse aggregate (solid)	1.87
Water (W/C)	0.65
Total (solid)	3.97

Multiplying the above by 62.4 times the appropriate values for specific gravity gives the following:

Weight proportions = 94:161:307
$W/C=0.65$ (by volume).

LOW WATER-CEMENT RATIO MAINTAINED USING FINE SANDS

By this procedure any mix, within a reasonable range, can readily be calculated provided the proper values of b/b_0 and W_c have been predetermined on the materials under investigation and, further, provided the slump is to remain constant. As can be seen from the example given above, the only unknown quantity in the mix is the amount of sand and this is determined by simple calculation. In these tests the water-cement ratio determined by calculation from the law of constant water per unit volume of concrete for a constant slump was not always the exact value needed to obtain the proper slump. However, it was possible in all cases to make the proper adjustment by slight changes in the ratio of water to sand, keeping the sum of the absolute volumes of the two ingredients constant. Even though the general law did not hold precisely in all individual cases, that is, to the third decimal, the following tabulation, which gives average values for the 25 combinations, will illustrate its accuracy.

Cement factor, sacks per cubic yard:	W_c
4.4	0.164
5.2	.163
6.0	.163
6.8	.163
7.2	.163

In connection with the procedure employed, that is, the use of the highest value of b/b_0 compatible with workability, it is interesting to note that for a given coarse aggregate and a given cement factor it was possible to use approximately the same water-cement ratio irrespective of whether a coarse or fine sand was used. For instance, in table 3, using a cement factor of 6.0 sacks per cubic yard, coarse aggregate No. 7 was used with sands 2, 3, 5, and 7. These sands varied in fineness modulus from 2.22 to 3.37. However, the total range in water-cement ratio was only 0.02 (from 0.73 to 0.75), demonstrating that, by proper proportioning, it is possible to maintain a low water-cement ratio when using fine sands.

The concrete was mixed in a laboratory mixer of the type shown in figure 1. In order to approximate field conditions, the coarse aggregate was handled in a saturated surface-dry condition and the sand in a wet condition, correction for the free water in the sand being made when computing the water-cement ratio. Test specimens consisted of 6- by 6-inch beams, 21 inches long. In all, 625 specimens—25 combinations of material, times 5 mixes, times 5 specimens per mix made on each of 5 different working days—were fabricated. All strength tests were made at 14 days, the specimens being tested by the third point method

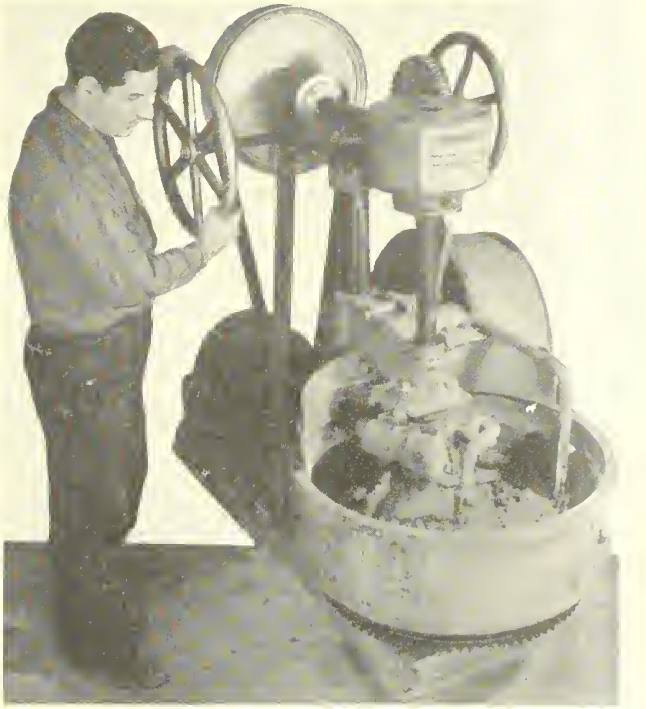


Figure 1.—Laboratory Mixer Used in Preparing Test Mixtures.

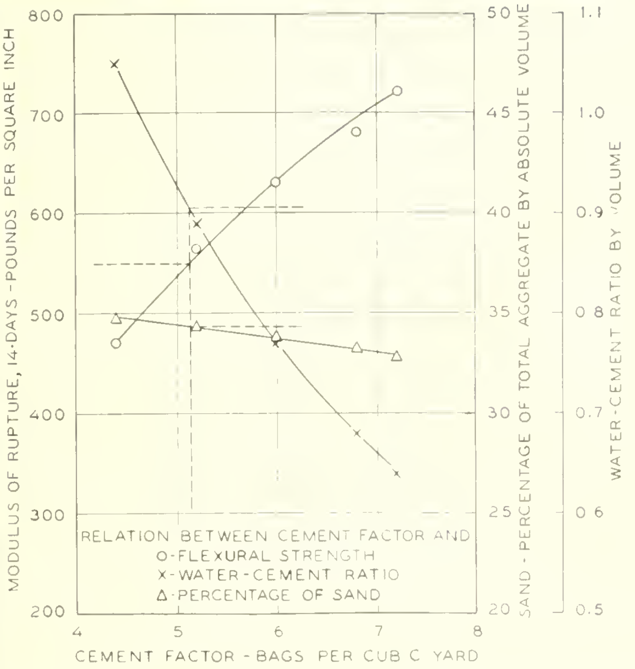
in accordance with A. S. T. M. Method C 78–39. The specimens were placed in the testing machine with the side, as molded, in tension.

The average flexural strengths for each combination of materials for each of the five different proportions are given in table 4. In discussing these results the various combinations of materials will be referred to by number. Thus, combination 2–4 refers to sand No. 2 combined with coarse aggregate No. 4. In order to illustrate the method of plotting the results for the purpose of determining the required cement factor for a specified modulus of rupture, two examples will be given.

CHART ENABLES COMPUTATION OF MIX DESIGN

Figure 2 shows a typical chart giving the relations between modulus of rupture and cement factor, water-cement ratio and cement factor, and percentage of sand and cement factor. Considering first the strength-cement factor relationship, it will be noted from the curve that the required amount of cement to produce a modulus of rupture of 550 pounds per square inch was 5.1 sacks per cubic yard. The corresponding water-cement ratio was 0.91 and the percentage of sand 34. From this type of chart a complete mix design could be computed for any strength specified within the range covered. In figure 2 for instance, the range in strength would be from about 500 to 700 pounds per square inch.

Frequently specifications for concrete contain a limiting value for water-cement ratio in order to assure durability. Assuming a maximum allowable value of 6 gallons per sack ($W/C = 0.80$ by volume), it will be observed from figure 2 that the required strength was obtained with a water-cement ratio of 0.91, which is more than allowable. From the water-cement ratio–cement factor curve it is seen that, in order to keep within the limits dictated by durability considerations, it would be necessary to use a cement factor of 5.8 sacks per cubic yard instead of 5.1 sacks, the resulting flexural strength being in excess of 600 pounds per square inch.



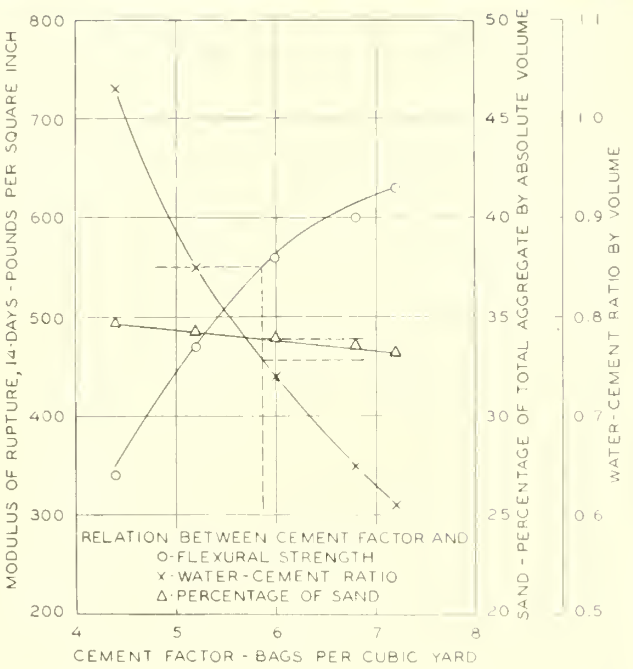
AGGREGATE	NUMBER	TYPE	SP GR	PERCENT ABS.	F M
FINE	2	SAND	2.63	0.50	2.22
COARSE	8	GRANITE	2.62	0.28	7.75

Figure 2.—Concrete Mix Design Chart for Combination 2–8.

Table 4.—*Average flexural strength, 14 days, of 6- by 6-inch beams tested on 18-inch span with third-point loading, for concrete mixes using North Carolina aggregates*

Aggregate		Modulus of rupture[1] for cement factor (sacks per cubic yard) of—				
Fine	Coarse	4.4	5.2	6.0	6.8	7.2
		Lb. per sq. in.	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
1	1	470	615	655	795	765
	2	460	575	655	695	755
	3	435	570	715	730	745
	10	380	425	510	535	545
2	4	455	510	540	630	655
	5	430	505	575	635	630
	6	475	570	620	640	615
	7	440	505	555	610	640
	8	470	565	630	680	720
	9	470	565	600	665	680
	11	490	500	545	635	635
	13	430	485	535	560	615
3	4	350	470	570	600	610
	5	340	470	560	600	630
	6	355	470	565	645	630
	7	365	445	575	610	625
	9	390	495	620	665	660
4	13	430	485	510	585	585
5	7	395	490	565	610	670
	9	395	510	615	675	695
6	4	425	490	530	630	620
	6	465	545	585	685	670
	11	480	515	575	635	655
7	7	395	490	565	630	620
	12	505	555	630	675	685
Average		428	513	584	642	654

[1] Each value is average of 5 tests.



AGGREGATE	NUMBER	TYPE	SP. GR	PERCENT ABS	F. M
FINE	3	SAND	2.63	0.40	2.75
COARSE	5	GRANITE	2.65	0.46	7.75

Figure 3.—Concrete Mix Design Chart for Combination 3–5.

Figure 3 gives the same type of data for a different combination of materials. The required cement factor as determined from the curve is 5.9 sacks, with a corresponding water-cement ratio of 0.75. In this case the strength is the governing factor while in the former case the maximum allowable water-cement ratio governs.

Charts similar to figures 2 and 3 were drawn for each of the 25 combinations of materials and the required cement factors obtained from the strength curves as illustrated. These cement factors are enumerated in table 5 and show values ranging from 4.9 sacks for combination 1–1, to 7.1 sacks for combination 1–10, or, for all practical purposes, from 5 to 7 sacks. In view of the fact that the same sand was used in both combinations, the strength differential in this case is a direct function of the concrete-making properties of the two coarse aggregates.

Table 5.—*Cement factor required for 550 pounds per square inch modulus of rupture, third point loading, for concrete mixes using North Carolina aggregates*

Fine aggregate	Coarse aggregate	Cement factor
		Sacks per cubic yard
1	1	4.9
	2	5.1
	3	5.1
	10	7.1
2	4	5.8
	5	5.7
	6	5.1
	7	6.0
	8	5.1
	9	5.0
	11	5.8
	13	6.2
3	4	5.8
	5	5.9
	6	5.9
	7	6.1
	9	5.5
4	13	6.4
5	7	5.9
	9	5.5
6	4	6.0
	6	5.3
	11	5.6
7	7	5.9
	12	5.0

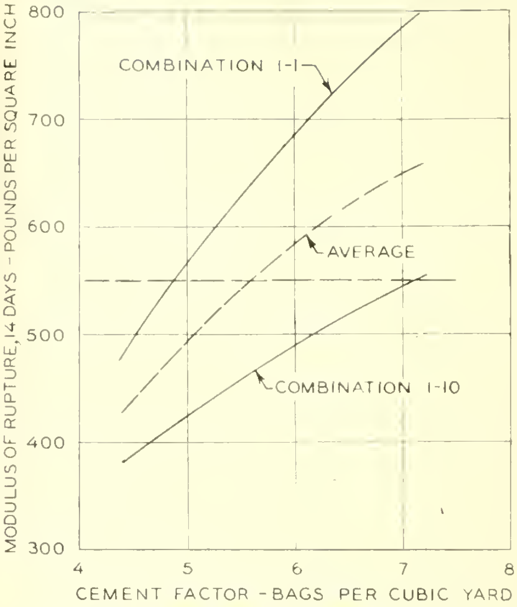


Figure 4.—Relation Between Cement Factor and Flexural Strength of Concrete. (Average Curve and Maximum Range.)

In order to illustrate better the effect of aggregate characteristics upon flexural strength, figure 4 is shown. Three flexural strength-cement factor curves are given, one for combination 1–1, one for combination 1–10, and one for the average of all 25 combinations of materials, thus representing the average and the extreme ranges for the entire series of tests. From the shapes of the two extreme curves it is seen that they tend to converge at the lower cement factors. This is to be expected because of the fact that for very lean mixes the strength of the mortar determines the flexural strength of the concrete. However, as the cement content is increased and the mortar becomes stronger, the quality of the coarse aggregate and the bond between mortar and coarse aggregate become important factors.

Coarse aggregate number 1 was a dolomite of excellent quality having a surface texture which gave good bond with the mortar. With this aggregate, the full strength of the mortar appeared to have been developed up to a cement factor of 7 sacks per cubic yard. Aggregate number 10 was a structurally weaker material so that the full strength of the mortar was not developed. If the water-cement ratio was the only controlling factor, the strengths of combination 1–10 should have been greater than those developed by combination 1–1 because of the fact that lower water-cement ratios were used (table 3). Notwithstanding this fact, combination 1–1 gave 26 percent greater strength for the leanest mix and 44 percent greater strength for the richest mix. At a cement factor of 6 sacks per cubic yard, one commonly used for pavement mixes, combination 1–1 gave 40 percent greater strength than combination 1–10. While in this particular instance the low strength was probably due to structurally poor material, this is not always the case. Frequently aggregates which are structurally strong give comparatively low flexural strengths in concrete because of the fact that the surface texture does not permit of sufficient bond to develop the full strength of the mortar.

(*Continued on p. 138*)

DETERMINATION OF THE KINEMATIC VISCOSITY OF PETROLEUM ASPHALTS WITH A CAPILLARY TUBE VISCOSIMETER

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **R. H. LEWIS**, Chemist, and **W. J. HALSTEAD**, Junior Chemist

IN many research investigations of bituminous materials the determination of consistency in poises or stokes is desirable since fundamental units give an accurate basis of comparison for different types of materials. Accurate comparisons are often very difficult to make from test values obtained by the use of empirical methods. In 1935, Messrs. Rhodes, Volkmann, and Barker[1] reported the development of a new viscosimeter of the capillary tube type for the determination of the consistency of bitumens. They found that this instrument could be used to measure, at the same temperature, the absolute viscosity of all grades of road tars. Since its introduction, this instrument has been used in several investigations. Reports of these investigations [2,3] indicate that the instrument is useful and gives very accurate results.

A study of the viscosimeter, the data of which are given in this report, was made to ascertain the accuracy and the scope of this instrument for determining the kinematic viscosity of asphaltic materials, more especially petroleum asphalts of the 50–60 and 85–100 penetration grades. The value of these results for determining the viscosity-temperature susceptibility was also studied. Since at the present time there is much interest in these problems, the results of this investigation are reported in detail for the benefit of those actively engaged in the testing of bituminous materials.

INSTRUMENT USES PRINCIPLE OF CAPILLARITY

The operation of the viscosimeter is described by Rhodes, Volkmann, and Barker[1], as follows:

> With this new instrument, viscosity is determined by timing the flow of the material under test through a capillary tube. Contrary to the classical method of Ostwald, which utilizes gravitation as the driving force, the material is made to ascend through the capillary tube. This is accomplished by immersing in the sample to be tested the lower end of a capillary tube, the upper end of which is connected with a partly evacuated reservoir. Two fixed points are chosen on the capillary tube and the time required for the passage of the air-sample interface between these points is noted. From the time, vacuum applied, the distance between points, and the radius of the bore of the capillary tube, the viscosity of the sample can be calculated.

The equation for calculating the viscosity from the observed data was derived by Volkmann, Rhodes, and Work[2] from Poiseuilles' equation for viscosity by capillary flow. This equation is:

$$\frac{\nu}{\Delta t}=\frac{gr^2}{8\left[\left(\frac{h}{\rho}+\lambda\right)\times\log_e\left[\frac{\left(\frac{h}{\rho}+\lambda\right)-l_1}{\left(\frac{h}{\rho}+\lambda\right)-l_2}\right]-(l_2-l_1)\right]}\text{ - - - -}(1)$$

where

ν=kinematic viscosity in stokes,
g=acceleration due to gravity (980 cm. per sec.^{2}),
r=radius of capillary bore (cm.),
h=vacuum in centimeters of water,
λ=length of capillary submerged in sample (cm.),
l_1=length of capillary filled with liquid at start of time interval (cm.),
l_2=length of capillary filled with liquid at end of time interval (cm.),
ρ=density of sample at test temperature (gm. per cm.^{3}), and
Δt=time of rise between l_1 and l_2 (sec.).

As explained by the earlier authors, [1] this equation is very difficult to use for extremely low values of the logarithmic expression. For this reason, they expanded the logarithmic expression into a series and obtained the equation in the following form:

$$\frac{\nu}{\Delta t}=\frac{gr^2}{8\left[\frac{l_2^2-l_1^2}{2\left(\frac{h}{\rho}+\lambda\right)}+\frac{l_2^3-l_1^3}{3\left(\frac{h}{\rho}+\lambda\right)^2}+\frac{l_2^4-l_1^4}{4\left(\frac{h}{\rho}+\lambda\right)^3}\right]}\text{ - - - -}(2)$$

In this study equation 2 has been simplified by neglecting all but the first two terms of the series in the denominator. Since λ is always controlled at 1.0 centimeter, the value $\lambda=1$ is used and the equation reduces to the following form:

$$\frac{\nu}{\Delta t}=\frac{gr^2}{4(l_2^2-l_1^2)}\left[\frac{h}{\rho}+1-\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_2+l_1)}+\frac{\left[\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_2+l_1)}\right]^2}{\frac{h}{\rho}+1+\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_2+l_1)}}\right]\text{ - -}(3)$$

A further approximation may be made by neglecting the last term in equation 3. The equation then becomes:

$$\frac{\nu}{\Delta t}=\frac{gr^2}{4(l_2^2-l_1^2)}\left[\frac{h}{\rho}+1-\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_2+l_1)}\right]\text{ - - - - - -}(4)$$

This equation may be expressed in the following form:

$$\frac{\nu_a}{\Delta t}=K\left(\frac{H}{\rho}\right)+C\text{ - - - - - - - - - - - - - - -}(5)$$

where

ν_a=kinematic viscosity in centistokes,
H=vacuum in centimeters of mercury, and
K, C=constants for chosen values of l_1 and l_2.

The constants K and C include all instrumental constants and conversion factors as shown in table 1. This table also shows values of K and C for the various height intervals used in this report.

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[1] New Viscosimeter for Bitumens Has Extended Range. Engineering News-Record, vol. 115, Nov. 21, 1935.

[2] Physical Properties of Coal Tars, by Volkmann, Rhodes, and Work. Industrial and Engineering Chemistry, vol. 28, June 1936.

[3] Consistency Measurements in the Coal Tar Industry, by Rhodes, Volkmann, and Barker. Symposium on Consistency, A. S. T. M., June 1937.

APPROXIMATE EQUATIONS MAKE POSSIBLE USE OF VARIOUS VALUES FOR l_1 AND l_2

The two approximations that have been made in deriving equation 4 have opposite effects. The omission of all but the first two terms of the series in the denominator of equation 2 tends to increase the computed values of $\nu/\Delta t$, while the omission of the last term in equation 3 tends to decrease these values. The increase in computed values caused by the first approximation will be greater for large values of l_2 and small values of l_1. For a given value of l_2, the decrease caused by the second approximation will be least for small values of l_1. Therefore, the greatest error caused by the use of the approximate equation 4 will result when the value of l_2 is large and that of l_1 is small.

Table 1.—*Equations for calculating viscosity, and values of* K *and* C *for various values of* l_1 *and* l_2

$$\frac{\nu}{\Delta t}=\frac{gr^2}{4(l_2^2-l_1^2)}\left[\frac{h}{\rho}+1-\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_1+l_2)}\right] \text{--------} (4)$$

or:

$$\frac{\nu_a}{\Delta t}=K\left(\frac{H}{\rho}\right)+C \text{------------------} (5)$$

where:

Length of capillary submerged in sample = 1.0 cm.

$$K=\frac{gr^2}{4(l_2^2-l_1^2)}\times D_{Hg}\times 100$$

$$C=\frac{gr^2}{4(l_2^2-l_1^2)}\left[1-\frac{2(l_2^2+l_2l_1+l_1^2)}{3(l_2+l_1)}\right]\times 100$$

ν = Kinematic viscosity in stokes,
ν_a = Kinematic viscosity in centistokes,
Δt = Time of rise between l_1 and l_2 (sec.),
r = Radius of capillary bore (0.0571 cm.),
g = Acceleration due to gravity (980 cm. per sec.^{2}),
l_1 = Length of capillary filled with liquid at start of time interval (cm.),
l_2 = Length of capillary filled with liquid at end of time interval (cm.),
h = Vacuum in centimeters of water,
H = Vacuum in centimeters of mercury.
ρ = Density of sample at test temperature (gm. per cm.^{3}),
D_{Hg} = Density of mercury at 25° C. (13.534 gm. per cm.^{3}).

l_1	l_2	K	C	l_1	l_2	K	C
2	3	216.20	−24.5	3	5	67.57	−15.4
2	4	90.09	−14.0	4	5	120.10	−31.2
2	5	51.48	−10.3	3	8	19.66	−7.1
2	6	33.78	−9.1	3	10	11.88	−5.4
3	6	40.04	−10.8	3	11	9.65	−4.8
4	6	54.05	−16.2	4	11	10.30	−5.4
5	6	98.28	−32.8	4	12	8.45	−4.8
3	4	154.50	−28.8				

In this study the largest interval used was $l_1=4$ centimeters, $l_2=12$ centimeters. In table 2 are shown comparisons of the values of $\nu_a/\Delta t$ obtained with the exact equation and the approximate equation 4. The exact calculations were made by equation 1 except where notation is made that equation 2 was used. For the purposes of comparison, these calculations have been carried to a greater number of significant figures than would ordinarily be employed in computing viscosities from the test results. Calculations are shown for the intervals $l_1=4$ centimeters, $l_2=12$ centimeters, the largest interval used, and $l_1=3$ centimeters, $l_2=6$ centimeters, one of the intervals more frequently used in this study. The values of H/ρ included vary from 1.00 to 7.31 and it is seen that for all values of H/ρ of 3 or more the maximum error is less than 0.5 percent, the error decreasing rapidly with increasing values of H/ρ. Although for values of H/ρ even as low as 2.20 the maximum error is only approximately 1 percent, for all values of H/ρ less than 3 it will generally be desirable to compute viscosities by equation 1, or to use a graph constructed with calculations made by this equation.

In this investigation, data on the specific gravity or density of the materials were available, and thus the kinematic viscosity has been reported. However, the absolute viscosity is desirable in many cases and for viscous asphaltic materials can be calculated within the accuracy of the method without the use of the density.

Table 2.—*Comparison of values of* $\frac{\nu_a}{\Delta t}$ *obtained with exact and approximate equations*

$\frac{H}{\rho}$	$l_1=4$ cm., $l_2=12$ cm.			$l_1=3$ cm., $l_2=6$ cm.		
	Values of $\frac{\nu_a}{\Delta t}$		$\frac{\nu_a}{\Delta t}$ (eq. 4) / $\frac{\nu_a}{\Delta t}$ (exact)	Values of $\frac{\nu_a}{\Delta t}$		$\frac{\nu_a}{\Delta t}$ (eq. 4) / $\frac{\nu_a}{\Delta t}$ (exact)
	Exact	Equation 4		Exact	Equation 4	
1.00	3.14	3.66	1.16561	28.98	29.19	1.00725
1.20	5.00	5.35	1.07000	37.03	37.20	1.00459
1.40	6.78	7.04	1.03835	45.07	45.21	1.00311
1.80	10.24	10.42	1.01758	61.13	61.22	1.00147
2.20	13.67	13.80	1.00950	77.16	77.24	1.00104
2.60	17.08	17.18	1.00585	93.18	93.25	1.00075
3.00	[1] 20.47	20.56	1.00440	[1] 109.22	109.27	1.00046
3.62	[1] 25.72	25.80	1.00311	[1] 134.07	134.11	1.00030
5.10	[1] 38.23	38.29	1.00157	[1] 193.26	193.28	1.00010
7.31	[1] 56.96	57.02	1.00105	[1] 282.02	282.04	1.00007

[1] Values computed by equation 2 with 6 terms in the series.

It has been shown (equation 5) that the equation for kinematic viscosity is:

$$\frac{\nu_a}{\Delta t}=K\left(\frac{H}{\rho}\right)+C$$

$$\nu_a=\frac{\mu}{\rho};\ \mu=\text{absolute viscosity in centipoises.}$$

thus

$$\frac{\mu}{\Delta t}=\rho\left[K\left(\frac{H}{\rho}\right)+C\right]$$

$$\frac{\mu}{\Delta t}=KH+\rho C\cdot$$

For high vacuums the constant C is small compared to KH and the difference between C and ρC would not affect the results appreciably since for bituminous materials the density is usually very close to 1 gram per cubic centimeter.

In making the tests on asphalts it was found much more satisfactory to vary the values for l_1 and l_2 with the general consistency of the material rather than use only the intervals 4 to 12 centimeters for materials of low viscosity and 2 to 4 centimeters for those of high viscosity, as suggested in the original report.[1]

The values for the height intervals most frequently used in this investigation were 3 to 6 centimeters for the higher temperatures, and 2 to 4 centimeters for the lower temperatures in testing the semisolid asphalts. For the test runs on the liquid materials the intervals 4 to 12 centimeters or 3 to 11 centimeters were generally used.

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[1] New Viscosimeter for Bitumens Has Extended Range. Engineering News-Record, vol. 115, Nov. 21, 1935.

The capillary tubes were calibrated by calculating the volume, and thus the diameter, from the weight of mercury necessary to fill the tube over a definite recorded length. These calibrations were made both at the beginning and end of the tests. The results are shown in table 3.

The standard diameter is 0.1142 centimeters. An equation for the correction factors for the differences from this standard was derived; it is:

$$\left(\frac{\nu}{\Delta t}\right)_1=\left(\frac{\nu}{\Delta t}\right)_0\left(1+\frac{\Delta d}{r_0}\right)$$

where

$\left(\frac{\nu}{\Delta t}\right)_1$ = correct value,

$\left(\frac{\nu}{\Delta t}\right)_0$ = value computed for standard diameter.

r_0 = standard radius, and

Δd = difference between actual and standard diameters.

This is an approximate equation that gives results differing from those given by the exact equation by negligible amounts.

TABLE 3.—*Initial and final diameters of capillary tubes*

Tube No.	Initial diameter	Final diameter
	Cm.	*Cm.*
1	0.1149	0.1150
2	.1144	.1141
3	----------	.1149

Using this equation, factors for each tube were found to be those given in table 4.

Since the maximum difference between the correct and computed value is approximately 1 percent, the precision of the determinations did not warrant making the correction.

APPARATUS MODIFIED TO GIVE MORE ACCURATE RESULTS

For this work the sample container supplied with the instrument was discarded and a test tube of 8-inch length and 1-inch inside diameter was used. This permitted use of a larger amount of material, and the clear vertical wall of the tube enabled better vision. In addition, sufficient test tubes were available so that the samples could be stored and repeated determinations at various temperatures could be made.

TABLE 4.—*Conversion factors for diameters of capillary tubes*

Tube No.	Conversion factor	
	Initial	Final
1	1.012	1.014
2	1.0035	.9982
3	----------	1.012

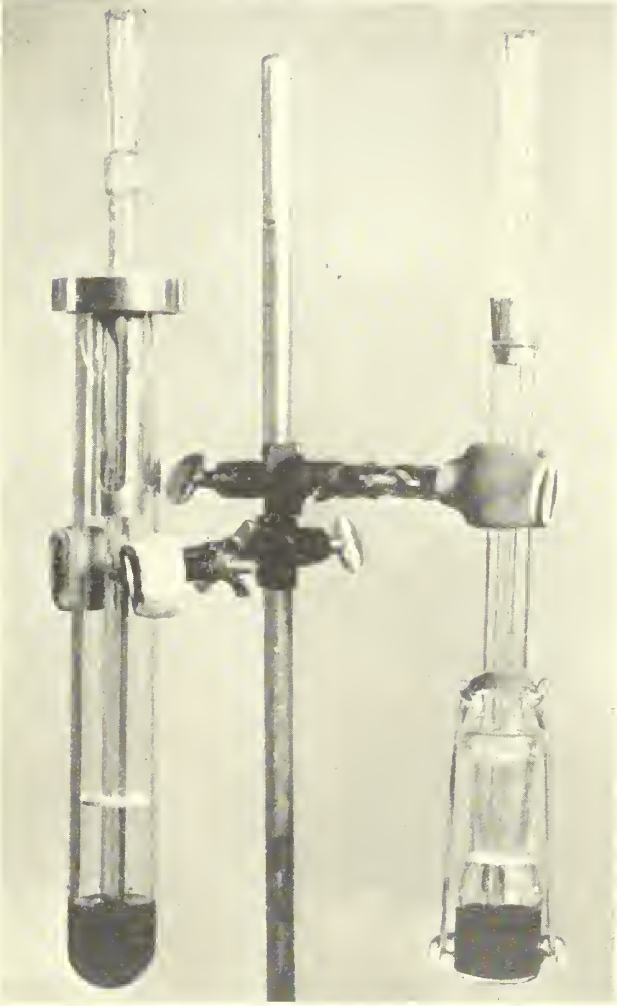

FIGURE 1.—MODIFIED TESTING UNIT (LEFT), AND UNIT SUPPLIED WITH INSTRUMENT (RIGHT).

It was found that the centering and depth of immersion of the tube were important factors in obtaining accurate results. In order to insure accuracy, a special holder which fits tightly in the top of the tube was made. It holds the capillary in place with a finger spring just tight enough to allow it to slide easily but not slip. In addition to this holder, a pasteboard disk cut to fit loosely in the test tube and tightly on the capillary, was placed on the tube approximately 3 centimeters above the surface of the sample. The use of these appliances made it possible to locate the capillary exactly in the center of the tube and to determine the depth of immersion accurately without difficulty. Figure 1 shows a photograph of the modified testing unit and the unit furnished with the instrument.

Accurate temperature control was obtained by the use of a standard thermometer placed in a test tube containing asphalt of the same type as the material under test. This gave a measure of the asphalt temperature at the center of the tube rather than the bath temperature.

A large enough portion of the sample to be tested to fill about 1 inch of the tube was heated and poured, then allowed to cool to room temperature for 1 hour. It was then placed in the bath maintained at the test temperature and allowed to stand for at least 1 hour before testing. The capillary tube was put in position at least 20 minutes before the first determination.

The reservoir was evacuated to the desired amount, the capillary connected to the system, and the stopcock opened. The time of rise through the distance

l_1 to l_2 was recorded with a stop watch. The mercury manometer for the determination of vacuum was read at the beginning and at the end of the test. Under ordinary circumstances the vacuum will remain constant throughout the test since the decrease in volume of the air space resulting from the rise of the asphalt is infinitesimal compared to the total volume of the reservoir.

The temperature, length of time, initial and final height, vacuum, and density of the material at test temperature were recorded. These data were then used to calculate the viscosity.

The capillary tubes were cleaned by heating in a bath of nitrobenzene maintained at approximately 100° C. by a boiling water bath. Air pressure was used to blow the asphalt out of each tube after it had softened sufficiently from the action of the heat and solvent. The tube was then washed clean in the same bath by alternate soaking and flushing, and the excess nitrobenzene removed with a current of air. The tube was allowed to cool slightly and then washed with ethyl ether and dried with air. Using this method, the tubes could be cleaned thoroughly in 3 to 5 minutes.

Check determinations were made on the same samples after the capillary tube had been cleaned and replaced. In order to insure complete equilibrium the capillary was allowed to stand in the sample at least 20 minutes before each test.

Samples that had remained undisturbed in the test tubes overnight or longer were preheated in an oil bath to 140–150° C. They were then allowed to cool to room temperature and the described procedure was followed. This preheating was found necessary in order to eliminate any effect of age hardening, except when the test temperature was 20 or more degrees above the softening point of the materials being tested. For these temperatures, tests showed no difference between results obtained on samples preheated and those placed directly in the bath.

The following four variables must be recorded for each determination: Temperature of test, vacuum (H), initial and final values of l_1 and l_2, and the time of rise (Δt). However, three of these are independent and are made constant for each determination; that is, the temperature, the values of l_1 and l_2, and the vacuum. The time of rise is the only dependent variable and there is no difficulty in recording the necessary data.

The viscosity can be calculated very easily by the use of the approximate formulas as derived. The time required is no more than that necessary when the chart supplied with the instrument is used, and direct calculations eliminate the necessity for interpolating values of the density. It also makes possible the use of more convenient values for l_1 and l_2.

FLOW GENERALLY UNIFORM THROUGHOUT LENGTH OF TUBE

Seventy-eight asphalts, which are representative of nearly every source and method of manufacture used in the United States, were studied in this investigation. Thirty-nine of these materials were of the 50–60 penetration grade and 39 were of the 85–100 penetration grade. Physical and chemical tests, both routine and special, have been made on all these asphalts, and are included in a report by Lewis and Welborn.[4]

Table 5 gives all available data on the source and method of refining these materials. Table 6 shows penetrations, softening points, and the kinematic viscosities at two temperatures of these asphalts.

TABLE 5.—*Source and method of refining asphalt cements*

Identification No.	Producer identification	Source of base petroleum	Method of refining
1	1	California, Coalinga field	Vacuum distillation.
2	2	California, San Joaquin Valley field.	Reduction and steam distillation.
3	3	do	Do.
4	4	do	Steam distillation in continuous-tube still.
5	5	California, Elk Hills field	Vacuum distillation.
6	6-A	Colombia	Vacuum distillation with pipe still.
7	7-A	Mexico, Ebano field	Straight steam distillation.
8	8	Mexico	
9	9	Mexico, Panuco field	Steam distillation in Trumble (pipe) still.
10	10	do	
11	11	do	Vacuum distillation in pipe still.
12	6-B	Mexico	Do.
13	12	Venezuela	Fire and steam distillation.
14	13-A	Venezuela, Mene Grande field.	Continuous distillation under subatmospheric pressure with steam.
15	13-B	do	Distilled in batch stills at atmospheric pressure with steam.
16	7-B	Venezuela	Straight steam distillation.
17	6-C	do	Vacuum distillation in pipe still.
18	14	do	Steam distillation.
19	15	Arkansas, Smackover field	Vacuum distillation at a low temperature.
20	16-A	do	Vacuum distillation, 89 melting point flux.
21	16-B	do	Vacuum distillation, 101 melting point flux.
22	17	Arkansas, Nevada County	Pipe still distillation unit and vacuum bubble tower.
23	18	Oklahoma, Cement and Walters field.	
24	19-A	Oklahoma	Vacuum distillation in pipe still, partially oxidized.
25	19-B	do	Do.
26	19-C	do	Do.
27	20-A	do[1]	
28	20-B	do	
29	21	Oklahoma, Healdton and Graham.	
30	22	Kentucky and Illinois	Fire and steam distillation, possibly blown.
31	23	Mexican-duo-sol-residuum from Oklahoma crude.	Steam distillation and air conversion in batch shell stills.
32	24-A	Kansas	Straight run, steam refined, vacuum process.
33	24-B	do	Produced from Winkler-Koch Shell still.
34	25-A	Wyoming	Fire and steam distillation.
35	25-B	Unknown	Do.
36	25-C	do	Do.
37	26-A	Mexico and domestic Gulf Coast.	
38	26-B	do	
39	27	Texas, Westbrook field	
40	28	Kentucky	Dubbs cracking process.

[1] Source assumed, considering the producer and from the interpretation of test results.

True viscous liquids have laminar or straight-line flow in a capillary tube. In deriving the formula for this instrument it is assumed that the materials under test are such viscous liquids. The theoretical formula is also derived from a consideration of the viscous resistance between parallel layers of the substance being tested, when these layers are moving at various speeds from zero at the outer wall of the tube to a maximum at the axis. This condition is fulfilled in most capillary viscosimeters by wetting the tube with the material under test before the determination is made. However, with opaque substances such as bituminous materials this is not practical, and the determination is made by timing the rise in a clean tube.

In order to ascertain the effect of these deviations from the theoretical conditions on the flow of the asphalts in the tube, tests were made in which the times of rise for various increments of height were recorded.

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[4] The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades. Annual meeting of Association of Asphalt Paving Technologists, Chicago, Ill., Jan. 1940. PUBLIC ROADS, March 1940, vol. 21, No. 1.

TABLE 6.—*Consistency of the asphalts studied*

Identification No.	50–60 penetration grade: Softening point	50–60 penetration grade: Penetration 100 gm. 5 sec., 25° C.	50–60 penetration grade: Kinematic viscosity At 65° C.	50–60 penetration grade: Kinematic viscosity At 85° C.	85–100 penetration grade: Softening point	85–100 penetration grade: Penetration 100 gm. 5 sec., 25° C.	85–100 penetration grade: Kinematic viscosity At 55° C.	85–100 penetration grade: Kinematic viscosity At 75° C.
	°C.		*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*°C.*		*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$
1	48.1	57	10.0	1.29	44.9	85	25.2	1.62
2	48.0	61	16.5	2.07	44.1	96	27.0	1.93
3	47.9	61	10.5	1.18	44.2	95	23.3	1.92
4	48.0	60	14.7	1.20	44.2	92	22.5	1.75
5	48.8	58	10.7	1.49	45.0	91	34.7	2.66
6	52.2	52	25.1	2.28	47.0	92	43.8	3.84
7	55.3	58	39.9	3.58	49.0	96	65.9	5.92
8	54.7	56	68.8	5.05	49.7	96	60.4	4.41
9	55.4	53	53.9	5.61	49.2	96	75.6	6.24
10	54.8	56	53.1	5.76	49.5	95	80.1	6.31
11	55.7	54	55.1	6.16	48.5	97	68.9	6.20
12	55.8	55	50.1	5.77	48.4	97	67.7	5.68
13	55.3	51	100.6	7.25	47.4	94	52.7	4.10
14	52.2	52	24.7	2.43	46.2	95	36.0	3.07
15	52.0	52	46.6	3.31	46.2	92	41.2	3.17
16	55.8	48	78.5	6.10	47.0	94	75.1	5.06
17	53.2	48	37.5	4.20	47.8	92	70.7	5.28
18	54.1	51	32.8	4.05	50.4	85	74.1	5.96
19	51.8	57	22.3	2.69	46.4	90	41.4	3.70
20	58.4	58	76.9	5.45	49.3	90	71.3	5.26
21	54.6	57	31.4	3.17	46.1	97	42.6	3.79
22	58.4	57	76.2	6.33	47.3	96	62.6	4.58
23	48.8	60	7.3	.76	44.4	91	26.3	1.77
24	55.1	54	63.3	4.73	47.7	94	----	----
25	52.8	58	30.4	3.02	48.0	94	34.6	2.99
26	54.0	53	45.4	2.89	48.6	84	54.5	3.99
27	55.2	49	39.2	3.79	46.6	93	59.1	4.32
28	52.0	58	24.6	2.53	46.3	92	32.5	2.66
29	----	----	----	----	45.2	92	25.1	1.88
30	55.1	48	53.3	4.02	46.9	90	59.8	4.20
31	56.2	59	62.2	5.11	49.3	93	79.6	5.98
32	53.1	49	29.9	3.15	49.5	85	45.1	3.73
33	52.8	46	16.8	1.02	48.4	83	46.6	2.50
34	53.4	58	46.3	3.69	46.9	94	57.0	4.07
35	50.8	57	20.0	1.74	46.9	96	38.4	3.03
36	51.9	55	28.9	2.56	49.6	92	84.2	4.66
37	55.8	52	54.9	4.76	48.0	96	59.0	4.29
38	55.4	55	79.0	5.24	49.5	95	111.2	6.02
39	53.8	47	40.5	3.11	46.8	86	47.7	3.44
40	50.5	50	23.8	1.46	45.0	87	34.3	2.45

These tests were made on the 85–100 penetration asphalts at 55° C. only. Asphalts 2 to 18, inclusive, with the exception of asphalt 14, were tested at the following intervals of height (in centimeters): 2 to 3, 2 to 4, 2 to 5, 2 to 6, 3 to 6, 4 to 6, and 5 to 6. Asphalts 19 to 40, inclusive, with the exception of asphalts 24 and 26, were tested at the following intervals (in centimeters): 2 to 3, 3 to 4, 4 to 5, and 5 to 6.

In general, at least two rounds of tests were made with each of these height intervals. The average, the maximum and minimum calculated value, the height intervals at which the maximum and minimum occurred and the maximum and average deviations from the mean for each round are shown in table 7.

All asphalts showed some deviation in the results, and there was considerable difference in the behavior of the various samples. The maximum deviation from the mean varied from zero for one round on sample 33 to as high as 8.4 percent for one round on sample 27. While a few of the asphalts showed very erratic results, as will be discussed later, in general there was no definite trend either in the height interval at which the maximum or minimum occurred or in the effect of the source or type of manufacture on results.

In general the maximum viscosities were obtained more frequently for the larger values of l_2 and the minimum viscosities for the lower values of l_2. However, this trend is not definite enough to draw any positive conclusions. There was no point in the tube which gave consistently high or low values. The average maximum deviation from the mean for group A was 1.2 percent, and for group B was 1.3 percent. The mean deviation averaged 0.5 percent for group A and 0.7 percent for group B.

TABLE 7.—*Effect of various values of l_1 and l_2 on the computed values for viscosity of the asphalts of the 85–100 penetration grade at 55° C.*

A. CALCULATIONS FOR INTERVALS OF 2 TO 3 CM., 2 TO 4 CM., 2 TO 5 CM., 2 TO 6 CM., 3 TO 6 CM., 4 TO 6 CM., AND 5 TO 6 CM.

Identification No.	Kinematic viscosity: Maximum	Kinematic viscosity: Minimum	Kinematic viscosity: Average	Interval: For maximum value	Interval: For minimum value	Deviation from mean: Maximum	Deviation from mean: Average
	Centistokes $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Cm.*	*Cm.*	*Percent*	*Percent*
2	27.2	26.7	27.0	2–4	2–5	1.1	0.6
	27.2	26.8	27.0	4–6	2–4	.7	.4
3	23.6	23.0	23.2	5–6	2–5	1.7	.9
	23.6	23.1	23.3	5–6	2–5	1.3	.6
4	22.8	22.2	22.5	2–3	2–5	1.3	.8
	22.8	22.4	22.6	2–4	5–6	.9	.5
5	33.8	33.1	33.4	5–6	2–5	1.2	.6
	35.3	34.8	35.0	5–6	2–5	.9	.4
	36.3	35.4	35.8	5–6	3–6	1.4	.7
6	44.8	43.3	44.1	5–6	2–3	1.8	1.0
	44.4	43.2	43.7	2–3	2–5	1.6	.7
	43.9	43.2	43.6	4–5	3–4	.9	.6
7	68.4	65.8	66.7	5–6	2–4	2.5	.9
	65.7	64.0	65.1	5–6	2–3	1.7	.7
8	60.8	60.0	60.3	2–3	2–5	.8	.4
	61.5	59.6	60.6	5–6	2–4	1.7	.7
9	75.2	74.5	75.0	4–6	2–3	.7	.2
	76.4	76.1	76.2	3–6	2–4	.3	.1
10	80.5	78.8	79.8	4–6	2–3	1.3	.7
	81.0	80.0	80.4	4–6	2–4	.8	.7
11	69.4	69.2	69.3	2–3	2–4	.1	.1
	68.9	68.2	68.5	5–6	2–3	.6	.3
12	66.9	66.4	66.6	5–6	2–5	.5	.3
	69.0	68.3	68.8	3–6	2–3	.7	.3
13	53.1	51.9	52.6	5–6	2–4	1.0	.7
	53.1	51.9	52.7	4–6	2–4	1.5	.6
15	41.8	40.8	41.2	2–3	4–6	1.5	.8
16	75.4	74.7	75.1	2–5	5–6	.5	.2
	75.8	74.0	75.1	2–4	2–3	1.5	.5
17	71.3	70.3	70.7	5–6	2–5	.8	.4
	71.1	70.3	70.6	5–6	3–6	.7	.3
18	74.7	72.9	73.6	5–6	4–5	1.5	.6
	76.3	72.6	74.6	5–6	2–3	2.7	1.2
Average						1.2	0.5

B. CALCULATIONS FOR INTERVALS OF 2 TO 3 CM., 3 TO 4 CM., 4 TO 5 CM., AND 5 TO 6 CM.

Identification No.	Kinematic viscosity: Maximum	Kinematic viscosity: Minimum	Kinematic viscosity: Average	Interval: For maximum value	Interval: For minimum value	Deviation from mean: Maximum	Deviation from mean: Average
19	42.0	40.3	41.2	3–4	2–3	2.2	0.6
	41.8	41.1	41.5	4–5	5–6	1.0	.6
	41.9	41.3	41.6	2–3	5–6	.7	.5
20	72.8	70.4	71.5	2–3	4–5	1.8	1.0
	71.5	70.7	71.2	4–5	3–4	.7	.4
21	43.0	42.5	42.7	3–4	4–5	.7	.5
	43.3	41.7	42.5	4–5	5–6	1.9	1.2
22	63.3	62.0	62.6	2–3	5–6	4.4	.8
	64.1	62.7	63.4	2–3	5–6	1.1	.5
	62.3	61.3	61.7	5–6	2–3	1.0	.5
23	26.6	26.2	26.5	3–4	5–6	1.1	.6
	26.4	26.6	26.2	3–4	4–5	.8	.4
25	34.7	34.1	34.5	5–6	2–3	1.2	.5
	34.3	33.7	34.0	4–5	2–3	[illegible]	.5
	35.8	35.1	35.5	2–3	4–5	1.1	.8
28	33.7	32.6	33.2	2–3	4–5	1.8	.8
	32.8	32.3	32.6	4–5	3–4	.9	.6
	32.0	31.3	31.6	5–6	3–4	4.3	.7
29	25.0	24.9	24.9	3–6	2–5	.1	.4
	25.5	25.3	25.3	5–6	2–3	.8	.2
30	60.3	58.9	59.7	2–3	3–4	1.3	.5
	61.7	58.5	60.0	2–3	3–4	2.8	2.0
	60.0	59.3	59.7	2–3	5–6	.7	.7
32	45.6	44.6	45.0	5–6	2–3	1.3	.5
	45.6	44.8	45.1	5–6	2–3	1.1	.4
33	44.4	44.4	44.4	3–4	5–6	0	0
	46.7	45.8	46.1	5–6	3–4	1.3	.6
	49.9	48.7	49.1	2–3	5–6	1.6	.7
34	57.1	55.4	56.1	5–6	2–3	1.8	.9
	55.2	53.2	54.0	4–5	2–3	2.2	.6
35	37.7	37.1	37.4	5–6	2–3	.8	.5
	38.8	38.0	38.5	4–5	2–3	1.3	.8
	39.9	38.8	39.3	5–6	2–3	1.5	1.1
36	84.9	81.9	83.2	5–6	3–4	2.0	1.2
	85.6	84.8	85.2	4–5	3–4	.5	.4
37	59.3	57.9	58.5	5–6	2–3	1.4	.7
	60.5	58.7	59.4	4–5	3–4	4.9	1.0
40	34.6	34.1	34.4	2–3	5–6	.9	.5
	34.5	33.5	34.1	3–4	4–5	1.8	.8
Average						1.3	.7

TABLE 7.—*Effect of various values of l_1 and l_2 on the computed values for viscosity of the asphalts of the 85–100 penetration grade at 55° C.*—Continued

C. SAMPLES SHOWING ERRATIC RESULTS

Identification No.	Kinematic viscosity			Interval		Deviation from mean	
	Maximum	Minimum	Average	For maximum value	For minimum value	Maximum	Average
	Centistokes $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Cm.*	*Cm.*	*Percent*	*Percent*
27	65.6	60.6	62.5	2–3	5–6	5.0	2.4
	61.9	57.1	59.4	2–3	5–6	4.2	2.9
	72.3	62.7	66.7	2–3	5–6	8.4	4.8
31	80.8	75.0	78.1	5–6	2–3	4.0	2.4
	82.6	80.0	81.1	5–6	2–3	1.8	1.1
	80.7	77.5	79.6	5–6	2–3	2.6	1.5
38	113.8	108.3	111.0	5–6	2–3	2.5	2.2
	115.6	107.3	111.4	5–6	2–3	3.8	2.4
39	47.8	47.2	47.5	2–3	5–6	.6	.2
	49.2	45.6	47.9	3–4	4–5	4.8	1.2

Samples 27, 31, 38, and 39 gave results showing much greater deviations from the mean than did the other materials. These samples are also peculiar as judged by the trend which the results showed. In all rounds sample 27 showed a progressive decrease in the calculated viscosity as the value of l_2 increased. For example, results on one round were: 65.6×10^{4} centistokes for 2 to 3 centimeters; 62.4×10^{4} for 3 to 4 centimeters; 61.5×10^{4} for 4 to 5 centimeters; and 60.6×10^{4} for 5 to 6 centimeters. For samples 31 and 38, the opposite condition was true—the calculated viscosity increased as the value of l_2 increased. For sample 31, one round showed 75.0×10^{4} centistokes for 2 to 3 centimeters, 77.6×10^{4} for 3 to 4 centimeters, 79.2×10^{4} for 4 to 5 centimeters, and 80.8×10^{4} for 5 to 6 centimeters. Sample 39 gave consistent results for one round, the maximum deviation from the mean being below the average; but for the other round there was a maximum deviation of 4.8 percent even though the average results of the two rounds checked very closely.

It is interesting to note that the materials known to be blends of different base asphalts had greater deviations from the mean than most of the other materials. Samples 31 and 38 were blends and showed marked peculiarities as already discussed. Samples 30 and 37, also blends, showed differences from the ordinary, but to a lesser degree. No definite information as to the source of the base petroleum or the method of manufacture was available for sample 27, but there is a possibility that it, too, was a blend.

These results indicate that the inside diameter of the tube is uniform since, if this were not true, there would be separate points at which the maximum and minimum values would be obtained. The accuracy of the formulas is also indicated by this lack of any systematic variation in results. In addition to this, the fact that the same asphalt shows maximum deviations varying appreciably between separate rounds on the same sample indicate that for this grade of asphalt the conditions of the test, the procedure, and the exactness of recording the data, cannot be expected to give an accuracy better than about 1 percent for calculations made with various height intervals.

All of the asphalts of the 50–60 grade were tested at 65° and 85° C., and all those of the 85–100 grade were tested at 55° and 75° C. These results are reported in table 6. Other tests were made on a selected group of the asphalts over a range of temperature from 45° to 90° C., as shown in table 8.

TABLE 8. *Kinematic viscosity of selected asphalts at various temperatures*

50-60 PENETRATION GRADE

Identification No.	Kinematic viscosity at—								
	45° C.	55° C.	60° C.	65° C.	70° C.	75° C.	80° C.	85° C.	90° C.
	Centistokes $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$	*Centistokes* $\times 10^{-4}$
3			21.6	10.5		3.41	2.53	1.18	
7		167	78.7	39.9	19.5		5.94	3.58	
13				100.6	59.1	28.23	12.20	7.25	
14		120	51.6	24.7	12.9		4.01	2.43	
15				46.6	22.8	11.70	5.64	3.31	
23			16.0	7.3		2.14		.76	
24		330		63.3		17.01	8.84	4.73	2.82
25		147		30.4		9.47	5.24	3.02	1.82
26		248		45.4	20.2	9.95	5.38	2.89	1.77
30				53.3	26.4	13.61		4.02	
31		270	104.4	62.2	26.5		8.12	5.11	
32		148		29.9	16.5	9.80	4.83	3.15	
33				16.8	7.0	3.40		1.02	.65
40			48.4	23.8	11.5	4.52	2.88	1.46	

85-100 PENETRATION GRADE

Identification No.	45° C.	55° C.	60° C.	65° C.	70° C.	75° C.	80° C.	85° C.	90° C.
3	134	23.3		6.2		1.92		0.64	
7	387	65.9		19.5		5.92		2.31	
13	364	52.7		14.6		4.10		1.54	
14	237	36.0		9.4		3.07		1.12	
15	278	41.2		10.8		3.17		1.15	
23	233	26.3		7.2		1.77		.69	
25	205	34.6		10.2		2.99		1.21	
30	345	59.8		15.2		4.20		1.62	
31	513	79.6		22.2		5.98		2.23	
32	315	45.1		12.8		3.73		1.43	
33	593	46.6		9.6		2.50		.82	
40	291	34.3		8.6		2.45		.81	

GREATER PRECISION OBTAINED FOR LIQUID PRODUCTS THAN FOR SEMISOLID ASPHALTS

These reported results are the average of two or three tests; two when there was close agreement, and three or more when the difference in results of the first two tests was rather large. Since it was of primary interest to gain a definite knowledge of the ability to get check results with this type of material, an analysis of the maximum percentage deviation from the mean is shown in table 9. The total number of samples tested at each temperature and the number of samples which had maximum deviation from the mean between various percentage limits are shown. The average maximum deviation for all tests, and for values below 6.1 percent, are also included.

The average maximum deviation from the mean for all tests on the asphalts of 50–60 penetration, excluding those values of 6.1 percent or greater, was 2.3 percent; and for the asphalts of 85–100 penetration was 1.4 percent. The asphalts of the 85–100 grade were tested after the changes in the apparatus had been made, as previously described. Therefore, it is believed that the average maximum deviation of 1.4 percent more nearly represents the accuracy of the test than the higher figure obtained with the 50–60 grade.

It will be noted that 72.8 percent of the tests on the 85–100 grade checked within 2 percent and 85.1 percent had maximum deviations of less than 3 percent. In general, with most asphalts of these grades the operator should be able to obtain checks within ±2 percent deviation from the mean, by running 3 or 4 trials. However, some asphalts exhibit peculiarities and the results of repeated trials will not check closely despite every precaution of the operator.

TABLE 9.—*Precision of test results obtained with asphalts of the 50–60 and 85–100 penetration grades*

50–60 PENETRATION GRADE

Test temperature, °C.	Number of samples tested	Number of samples with maximum deviation from the mean between— 0–1.0 percent	1.1–2.0 percent	2.1–3.0 percent	3.1–4.0 percent	4.1–6.0 percent	6.1+ percent	Average maximum deviation For all values	For values less than 6.1
								Percent	*Percent*
55	4	3	0	1	0	0	0	0.9	0.9
60	12	0	1	4	4	1	2	3.5	3.0
65	39	14	8	8	4	5	0	2.1	2.1
70	10	4	2	1	0	1	2	2.6	1.5
75	11	2	3	2	1	0	3	3.5	1.8
80	11	5	1	2	2	1	0	1.9	1.9
85	39	13	10	7	2	3	4	2.2	1.7
90	4	2	1	0	0	1	0	1.8	1.8
Total	130	43	26	25	13	12	11		
Percentage		33.1	20.0	19.2	10.0	9.2	8.5		
Average								2.4	2.3

85–100 PENETRATION GRADE

Test temperature, °C.	Number of samples tested	0–1.0 percent	1.1–2.0 percent	2.1–3.0 percent	3.1–4.0 percent	4.1–6.0 percent	6.1+ percent	For all values	For values less than 6.1
45	12	3	2	3	2	2	0	2.5	2.5
55	39	24	8	4	1	1	1	1.4	1.1
65	12	5	2	0	1	2	2	2.7	1.6
75	39	19	13	4	2	1	0	1.2	1.2
85	12	5	2	3	1	1	0	1.7	1.7
Total	114	56	27	14	7	7	3		
Percentage		49.1	23.7	12.3	6.15	6.15	2.6		
Average								1.6	1.4

The most striking example of this erratic behavior was sample 33 of the 85–100 penetration grade. This material was a cracked product which showed a positive spot in xylene when tested according to the Oliensis method. The test results on this sample are given in table 10.

Sample 27 of the 85–100 penetration grade, which showed the greatest deviation in the same round (table 7), also gave the greatest difficulty in obtaining check results at 55° C. Six tests were made, the results varying as follows: 52.6×10^4; 58.8×10^4; 54.8×10^4; 62.5×10^4; 59.4×10^4; 66.7×10^4. The average was 59.1×10^4. The maximum deviation from the mean is 12.6 percent. The results at 75° C. for this sample did not show this same irregularity but checked within 0.6 percent deviation from the mean.

TABLE 10.—*Results of tests on sample 33*

Test temperature	Kinematic viscosity determination— No. 1	No. 2	No. 3	No. 4	Average	Maximum deviation from mean
° C.	*Centistokes* $\times10^{-4}$	*Centistokes* $\times10^{-4}$	*Centistokes* $\times10^{-4}$	*Centistokes* $\times10^{-4}$	*Centistokes* $\times10^{-4}$	*Percent*
5	587.9	563.6	623.9	[1] 464.6	593.4	5.1
5	44.39	46.13	49.14		46.6	5.6
5	[1] 12.28	10.44	8.85	9.89	9.6	8.3
5	[1] 3.000	2.506	2.494		2.50	.3
5	.80	.84			.82	2.5

[1] Test not included in average.

For testing the asphalts of the 85–100 penetration grade, the greatest average maximum deviation for all values varying less than 6.1 percent was 2.5 percent and this value was obtained for tests made at 45° C., the lowest temperature used for this grade of material. At the lowest test temperature used for the 50–60 penetration asphalts, which was 55° C., only seven asphalts were tested and on three of these only one test was made. The four that were checked had an average maximum deviation from the mean of only 0.9 percent. However, the average maximum deviations from the mean for the 50–60 grade asphalts tested at 60° C. and 65° C., excluding all values of 6.1 percent or greater, were higher than those values obtained at the higher temperatures. These results indicate that the instrument is less accurate for the determination of high viscosities of the magnitude obtained near temperatures of the softening points of the asphalts.

A number of liquid asphaltic materials of the rapid-, medium-, and slow-curing types, as well as two asphalts of approximately 200 penetration, also were tested for kinematic viscosity with this instrument. All of these materials were tested at 35° C., the temperature used in the work reported by Rhodes, Volkmann, and Barker.[1] Eleven samples were tested also at 25° C. and 50° C. The results of these tests are shown in table 11.

The viscosities of these materials at the test temperature of 35° C. ranged from 513 centistokes for the RC material to 138×10^4 centistokes for one of the 200 penetration asphalts. As in the case of the work done on the harder asphalts, two or more determinations were made on the same sample, and the mean of these results is the value reported for the viscosity. The maximum percentage deviation for the tests on each sample is also shown in table 11. It will be noted that more consistent results were obtained with these less viscous materials than with the semisolid asphalts. The greatest maximum deviation from the mean was only 2.6 percent and, out of 49 tests, only 7 had maximum deviations from the mean of 2 percent or more. Thirty-five samples checked within 1 percent or less. Test values widely different from the mean, as reported for some of the asphalts, were not obtained with these materials. In most cases three tests were made for each sample, and all the results were used to compute the average. The tests made on these materials indicate that the instrument has a relatively wide range, and that the consistency of a large number of asphaltic materials can be determined adequately at a temperature of 35° C.

VISCOSITY-TEMPERATURE DATA PLOTTED TO GIVE A STRAIGHT LINE

The data shown in table 6 on the kinematic viscosities of all the asphalts of each grade serve to emphasize the known fact that asphalts having essentially the same consistency at 25° C. may have widely different consistencies at higher temperatures. For example, the viscosity of the 50–60 penetration asphalts at 65° C. ranged from a minimum of 7.3×10^4 centistokes for sample 23 to a maximum of 100.6×10^4 centistokes for sample 13. These asphalts had penetrations at 25° C. of 60 and 51, respectively. The same relative difference is shown for the tests made at 85° C. While the maximum difference is not as great with the 85–100 grade, the viscosities ranged from 22.5×10^4 to 111.2×10^4 centistokes at 55° C., and from 1.62×10^4 to 6.31×10^4 centistokes at 75° C.

Since these differences are known to exist, knowledge

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[1] New Viscosimeter for Bitumens Has Extended Range. Engineering News-Record, vol. 115, Nov. 21, 1935.

TABLE 11.—*Viscosity determinations for various types of asphaltic materials*

Type	Grade designation [a]	Routine consistency determination				Tests at 25° C.		Tests at 35° C.		Tests at 50° C.	
		Furol viscosity		Engler specific viscosity at 100° C.	Penetration 100 gm., 5 sec. at 25° C	Kinematic viscosity	Maximum deviation from mean	Kinematic viscosity	Maximum deviation from mean	Kinematic viscosity	Maximum deviation from mean
		At 50° C.	At 60° C.								
		Seconds	*Seconds*			*Centistokes*	*Percent*	*Centistokes*	*Percent*	*Centistokes*	*Percent*
RC	1	108						513	1.0		
RC	2	320				794×10	1.1	262×10	2.3	584	0.6
RC	3		292					480×10	.4		
RC	3		325					532×10	.5		
RC	3		310					460×10	.7		
RC	3		299					505×10	.9		
RC	3		307					454×10	.2		
RC	3		286					453×10	.6		
RC	3		313					485×10	.3		
RC	3		325					497×10	.2		
RC	3		380					629×10	.2		
RC	3		383					519×10	.1		
RC	3		377					592×10	.5		
RC	3		340					588×10	.5		
RC	3		364					608×10	1.0		
RC	4		631			133×10^3	1.1	277×10^2	.4	430×10	2.0
MC	2	213				588×10	.8	186×10	2.0	487	.4
MC	2	234				101×10^2	.7	222×10	.4	554	2.2
SC	2		178					442×10	.1		
SC	2		196					475×10	1.7		
SC	2		170			168×10^2	.4	416×10	1.2	858	1.0
SC	([b])		225			300×10^2	1.3	657×10	.3	126×10	.8
SC	([b])		247			785×10	2.6	355×10	.8	106×10	1.1
SC	([b])		231			964×10	.4	378×10	.8	103×10	.4
SC	3		366			810×10^2	.7	153×10^2	1.0	241×10	.4
SC	4			21.7				698×10^2	.4		
SC	4			16.2				366×10^2	.3		
AP	([c])				202	178×10^5	([d])	108×10^4	1.0	761×10^2	2.2
AP	([c])				212	183×10^5	([d])	138×10^4	.2	884×10^2	2.6

[a] Asphalt Institute Construction Series Number 51, Jan. 1, 1940, except as noted in footnote *c*.
[b] Falls between grades 2 and 3.
[c] Penetration grade, 200–250.
[d] Only one test made.

of the change of consistency with temperature is very desirable. This problem has recently received the attention of many investigators. In order to determine the suitability of the instrument for obtaining these data, tests were made over a range of temperature for a selected group of asphalts. Fourteen of the 50–60 grade asphalts and 12 of the 85–100 grade were tested at temperatures varying from slightly below the softening points of the materials to 90° C. Table 8 shows the results of these tests, which range from approximately 6,000 to 6,000,000 centistokes.

While it is possible to obtain higher viscosities than here reported, the time of rise becomes so long that it is not practical to run tests for viscosities higher than 6,000,000 centistokes. For this value the time of rise from 2 to 3 centimeters is approximately 12 minutes at the maximum vacuum obtainable. In addition, the time of rise to the starting point is several minutes.

In tests run for other investigations, results as high as 31,000,000 centistokes were obtained. However, the time required for one test was approximately 3 hours. Inability to obtain check tests and the time required in making each determination renders the instrument unsuitable to use for such high viscosities.

It is also possible to obtain viscosities much lower than the 6,000 centistokes reported for these asphalts at 90° C., but the high temperature necessary for these grades of asphalt makes it difficult with the same procedure and bath normally used. With additional heating coils and suitable liquids for the thermostat, it would be possible to extend the range to higher temperatures than here used, but for the purpose of this investigation this was not considered necessary.

Graphical presentation of results is very desirable for viscosity-temperature data, and many relations have been proposed so that the curve when plotted becomes a straight line. Several investigators[5] working independently have shown that for a liquid of definite chemical composition the viscosity should change with temperature according to the formula:

$$\mu = Ae^{B/T}$$

where

μ = absolute viscosity,
T = absolute temperature,
e = constant (base of Napierian logarithms), and
A, B = constants.

When this formula is expressed in logarithmic form it becomes:

$$\log \mu = \log A + \left(\frac{B}{T}\right) \log e$$

Thus a straight line should be obtained when log μ is plotted against the reciprocal of the absolute temperature.

The data shown in table 8 were plotted in this manner except that the kinematic instead of absolute viscosity was used. However, the density of the asphalts is so close to 1 gram per cubic centimeter at these test temperatures that there is no essential difference between the curves plotted with either value.

Figures 2 and 3 show these curves for each grade of asphalt. As will be noted, the curvature of the line increases as the temperature approaches the softening point of the asphalt. As will be shown later, there is a possibility that the viscosities obtained with the instrument at or near temperatures of the softening point are higher than the true viscosities because of the limitations of the instrument. Consequently, the curvature

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[5] Elasticity, Plasticity and Structure of Matter, by Houwink. Cambridge University Press (1937), p. 38. See also. E. N. da C. Andrade, Nature, vol. 125 (1930) p. 580. S. E. Sheppard, J. Rheology, vol. 1 (1930), p. 349.

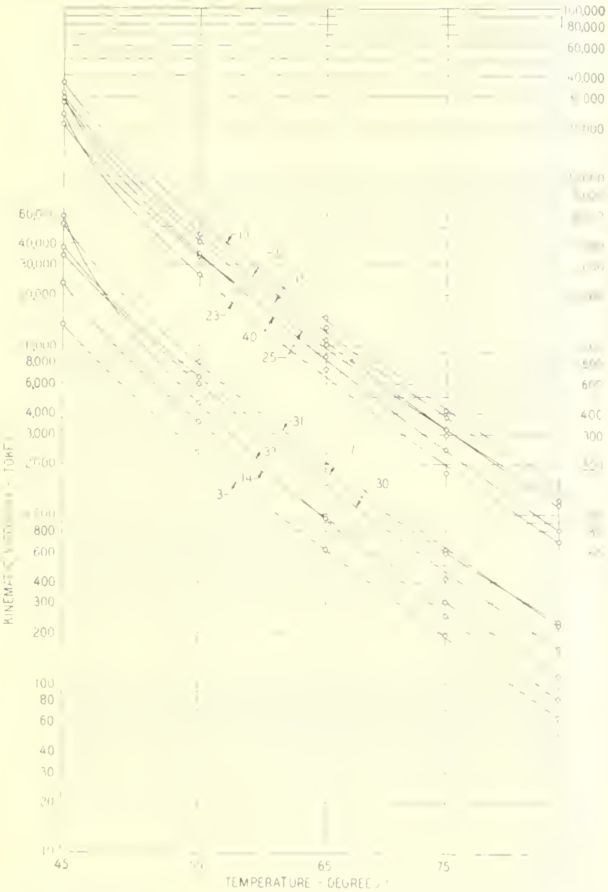

FIGURE 2.—RELATION BETWEEN LOG VISCOSITY AND RECIPROCAL OF ABSOLUTE TEMPERATURE FOR THE 85–100 PENETRATION ASPHALTS.

shown in the graph may be somewhat exaggerated. The shape of each curve is essentially the same, although some show slightly greater curvature than others.

Although asphalts are not liquids of a definite chemical composition, and in some cases exhibit plastic properties, these curves are of interest since they take into consideration the physical characteristics of the asphalts and no empirical constants have been introduced. It is believed that in a study of the deviations from a true liquid, or the changes in susceptibility over a wide range of temperature, these curves would be very valuable. However, from a practical standpoint they have no direct application and they cannot be used to calculate a susceptibility factor which would hold over an appreciable range of temperature.

As discussed by Rhodes, Volkmann, and Barker,[3] an empirical relationship between viscosity and temperature has been established. It is:

$$\log\left(\frac{\log(\nu_{a1}+0.8)}{\log(\nu_{a2}+0.8)}\right)=m\times\log.\frac{T_2}{T_1}$$

where ν_{a1} and ν_{a2} are the kinematic viscosities in centistokes at the absolute temperatures T_1 and T_2, respectively. Ubbelohde, Ullrich, and Walther[6] found this relationship to be the most accurate expression for the viscosity-temperature data for coal tars, and the same formula is used by the American Society for Testing Materials as the basis for the construction of the tentative standard viscosity-temperature charts for liquid petroleum products.[7]

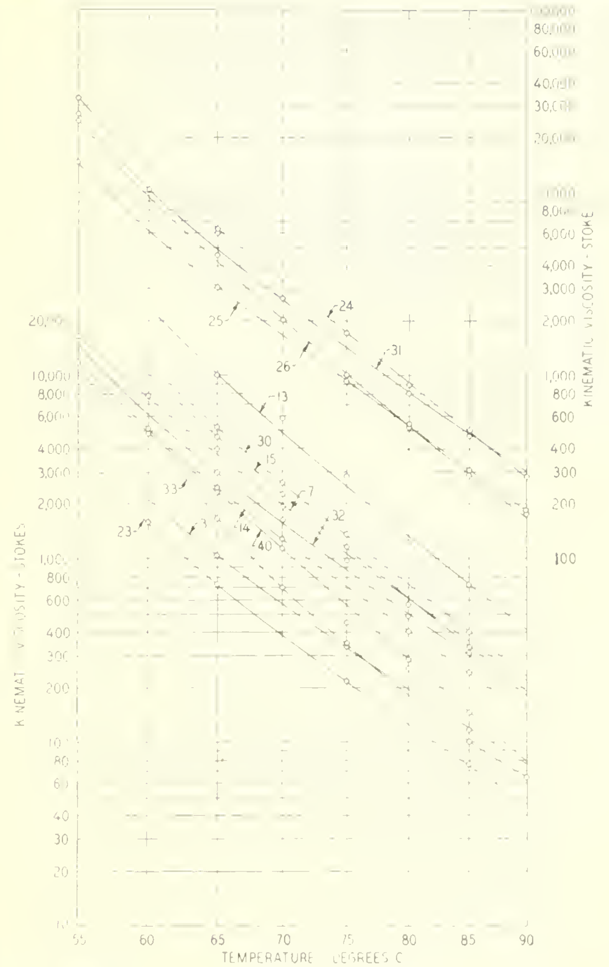

FIGURE 3.—RELATION BETWEEN LOG VISCOSITY AND RECIPROCAL OF ABSOLUTE TEMPERATURE FOR THE 50–60 PENETRATION ASPHALTS.

SLOPE OF VISCOSITY-TEMPERATURE CURVE DIFFERS FROM V. T. S. COEFFICIENT BY A CONSTANT FACTOR

For high viscosities the figure 0.8 added to the kinematic viscosity has no significance, and thus, if the double logarithm of the kinematic viscosity in centistokes is plotted against the logarithm of the absolute temperature, a straight line should be obtained.

The data in table 8 were plotted in this way and several typical curves are shown in figures 4 and 5. With the exception of some of the viscosities determined at or very close to the temperature of the softening point of the materials under test, all the values fall close to the straight line. If the viscosities at temperatures close to those of the softening point deviate from the straight line they always fall above it. No attempt was made to determine the cause of this deviation since no other instrument to measure absolute viscosities of this magnitude was available. This deviation may be caused by a change in the susceptibility of the asphalts at temperatures near the softening point, or because of

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[3] Consistency Measurements in the Coal Tar Industry, by Rhodes, Volkmann, and Barker. Symposium on Consistency, A. S. T. M., June 1937.

[6] Beitrag zur Kennzeichnung von Teeren und Bitumen auf Grund der Abhängigkeit iher Viskosität von der Temperatur, Oel und Kohle, Veneinigt mit Endoel und Teer, vol. 11, No 36, Sept. 22, 1935, pp. 684–690.

[7] Tentative viscosity-temperature chart for liquid petroleum products. A. S. T. M. D341–39T.

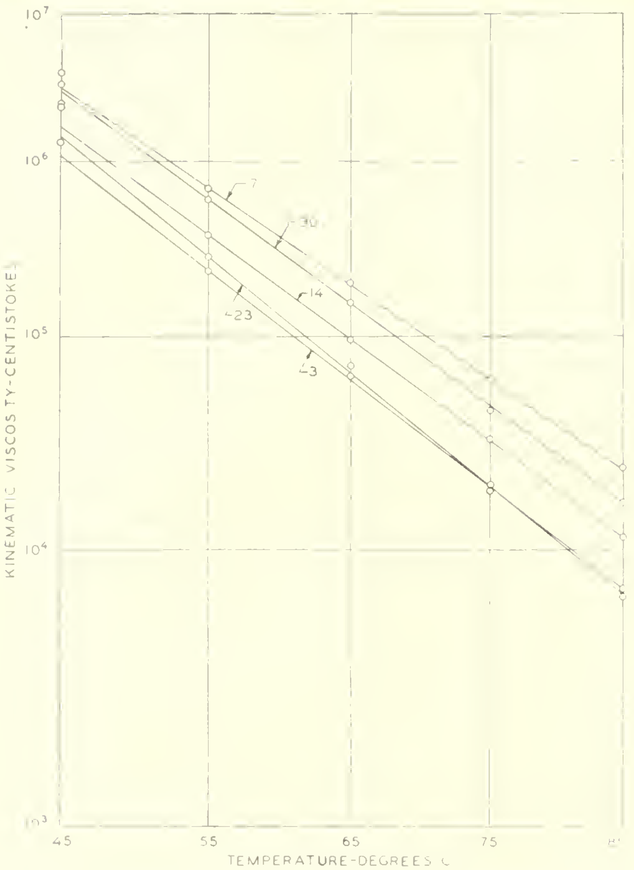

Figure 4.—Relation Between Log Log Viscosity and Log Absolute Temperature for Selected 85–100 Grade Asphalts.

the inability of the instrument to give accurate results at these high viscosities. However, no definite conclusions can be drawn.

The slope m of the viscosity-temperature curve can readily be calculated from the viscosity at two temperatures or from the graph when plotted. The equation for the slope may be expressed as follows:

$$m=\frac{\log \log \nu_{a1}-\log \log \nu_{a2}}{\log T_1-\log T_2}$$

where ν_{a1} and ν_{a2} are the kinematic viscosities in centistokes at the absolute temperatures T_1 and T_2 respectively.

The value of the slope m was calculated for all the asphalts using the viscosities at 65° C. and 85° C. for the 50–60 penetration grade and the viscosities at 55° C. and 75° C. for the 85–100 penetration grade. These data are shown in table 12. While the sign of these slopes is always negative, this sign only indicates the direction of the slope and is not shown in the table.

These slopes differ by a constant factor from the "viscosity-temperature-susceptibility" (V. T. S.) coefficient which has been proposed by H. G. Nevitt and L. C. Krchma[8] as a temperature susceptibility index, and which these authors claim gives more useful information than the various susceptibility factors based on empirical consistency measurements. The V. T. S. coefficient for these asphalts can be calculated by multiplying the slopes as given in table 12 by the factor 0.221. The V. T. S. coefficient also can be determined directly from the A. S. T. M. viscosity-temperature chart when the data are plotted, since it is equal to the tangent of the measured angle that the plotted line makes with the temperature axis.

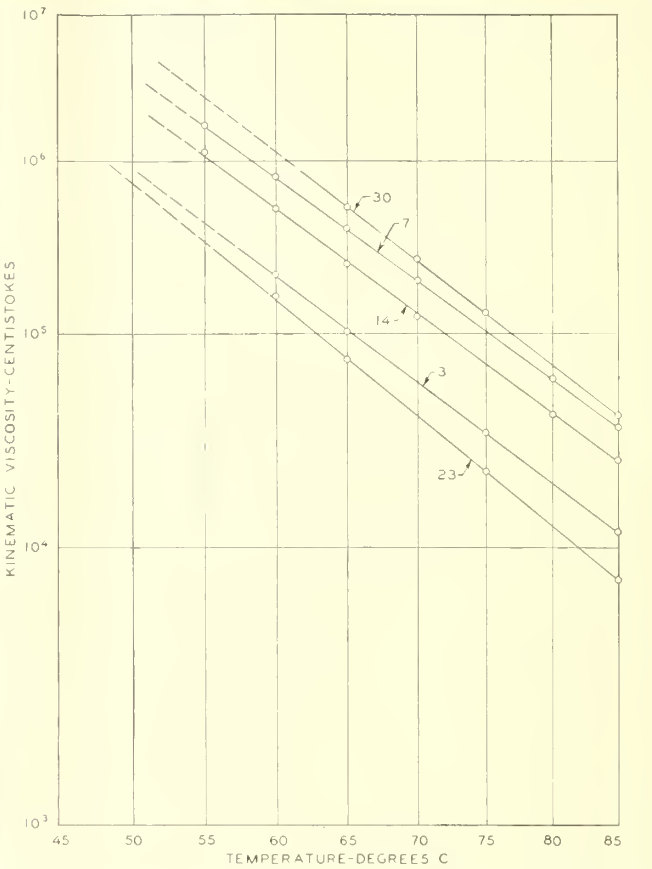

Figure 5.—Relation Between Log Log Viscosity and Log Absolute Temperature for Selected 50–60 Grade Asphalts.

Table 12.—*Calculated slope*[1] *of the log. log. kinematic viscosity–log. absolute temperature curves*

Identification No.	Slope		Identification No.	Slope	
	50-60 penetration grade	85–100 penetration grade		50–60 penetration grade	85–100 penetration grade
1	3.40	4.21	21	3.45	3.49
2	3.30	4.00	22	3.52	3.69
3	3.64	3.81	23	3.92	4.12
4	4.11	3.92	24	3.75	----------
5	3.24	3.80	25	3.51	3.25
6	3.72	3.51	26	4.17	3.73
7	3.60	3.35	27	3.48	3.70
8	3.75	3.70	28	3.52	3.71
9	3.27	3.44	29	----------	3.95
10	3.21	3.50	30	3.79	3.76
11	3.15	3.34	31	3.60	3.57
12	3.13	3.45	32	3.42	3.59
13	3.67	3.64	33	4.61	4.29
14	3.59	3.61	34	3.75	3.75
15	3.93	3.74	35	3.88	3.72
16	3.62	3.76	36	3.72	4.03
17	3.25	3.62	37	3.55	3.71
18	3.13	3.92	38	3.87	3.97
19	3.27	3.49	39	3.85	3.79
20	3.78	3.63	40	4.44	3.92

[1] Calculated from kinematic viscosity data given in table 6.

The viscosity values obtained on the 50–60 and 85–100 penetration grade asphalts at the temperatures used in this report could not readily be correlated with

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[8] The Effect of Temperature on the Consistency of Asphalts—the Viscosity Temperature Susceptibility Coefficient as an Index. Industrial and Engineering Chemistry, Analytical Edition, vol. 9, No 3, pp. 119–122 (1937).

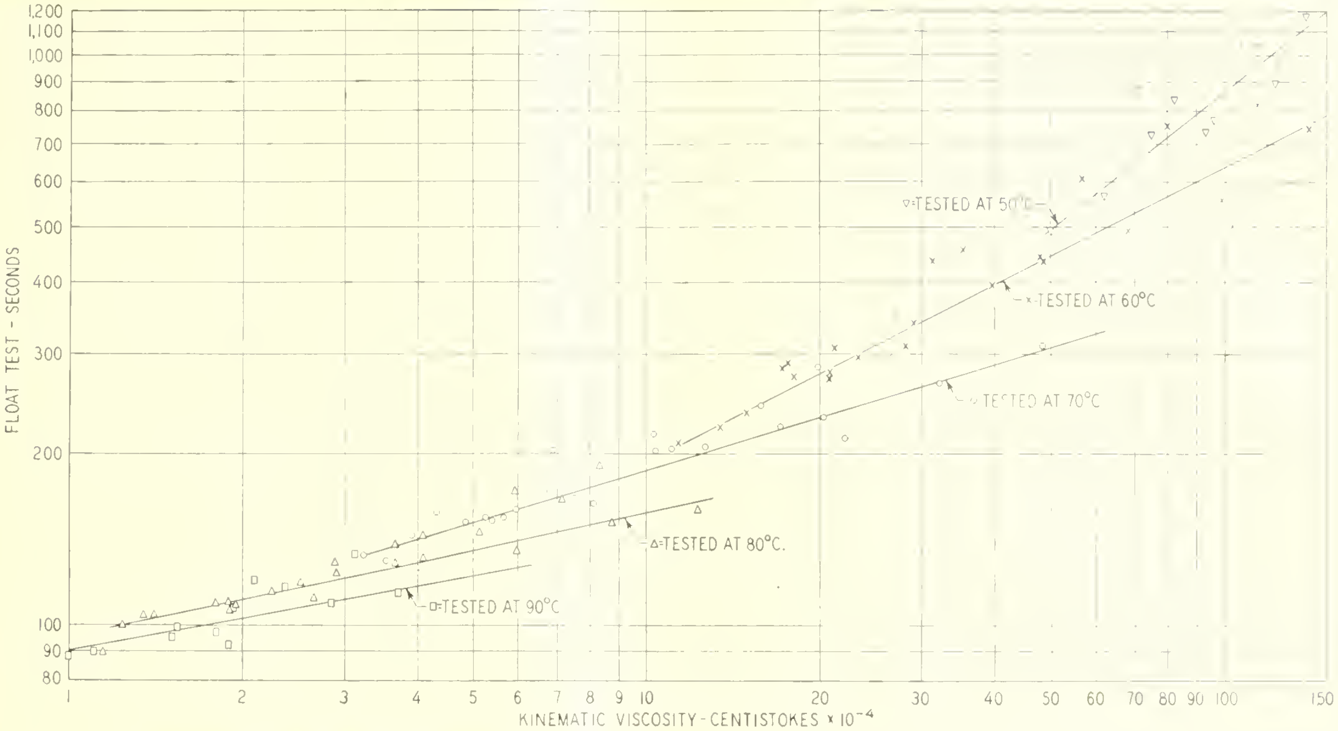

FIGURE 6.—RELATION BETWEEN KINEMATIC VISCOSITY AND FLOAT TEST AT VARIOUS TEMPERATURES.

TABLE 13.—*Float test and interpolated viscosity at various temperatures*

50–60 PENETRATION GRADE

Identification number	Tests at 50° C.		Tests at 60° C.		Tests at 70° C.		Tests at 80° C.		Tests at 90° C.	
	Float test	Kinematic viscosity	Float test	Kinematic viscosity	Float test	Kinematic viscosity	Float test	Kinematic viscosity	Float test	Kinematic viscosity
	Seconds	*Centistokes* ×10⁻⁴	*Seconds*	*Centistokes* ×10⁻⁴	*Seconds*	*Centistokes* ×10⁻⁴	*Seconds*	*Centistokes* ×10⁻⁴	*Seconds*	*Centistokes* ×10⁻⁴
3	------	------	308	21. 6	160	6. 0	108	2. 53	79	0. 73
7	------	------	751	78. 7	285	19. 5	173	5. 94	120	2. 11
13	------	------	858	241. 0	310	59. 1	160	12. 20	114	3. 72
14	------	------	443	51. 6	206	12. 9	131	4. 01	96	1. 51
15	------	------	504	102. 7	214	22. 8	135	5. 64	93	1. 89
23	------	------	237	16. 0	144	3. 9	100	1. 24	66	. 46
24	------	------	748	139. 5	267	32. 2	152	8. 84	109	2. 82
25	------	------	493	68. 3	224	17. 2	132	5. 24	97	1. 82
26	------	------	558	99. 5	232	20. 2	144	5. 38	98	1. 77
30	------	------	821	113. 8	300	26. 4	167	7. 13	117	2. 37
31	------	------	1, 067	104. 4	352	26. 5	193	8. 12	134	3. 07
32	------	------	606	56. 9	244	16. 5	146	4. 83	108	1. 93
33	------	------	438	31. 3	204	7. 0	123	2. 92	83	. 65
40	------	------	438	48. 4	204	11. 5	129	2. 88	88	. 94
85–100 PENETRATION GRADE										
	492	50	210	11. 4	133	3. 22	90	1. 11	------	------
	1, 187	137	457	35. 2	217	10. 30	139	3. 68	------	------
13	893	122	310	28. 2	168	7. 50	118	2. 52	------	------
14	729	75	273	18. 2	155	5. 26	109	1. 80	------	------
15	734	93	271	20. 9	155	5. 66	109	1. 89	------	------
23	567	62	223	13. 5	130	3. 55	89	1. 14	------	------
25	874	71	290	17. 7	153	5. 90	106	1. 89	------	------
30	966	124	340	29. 1	164	8. 11	112	2. 67	------	------
31	1, 276	167	394	39. 6	205	11. 10	128	3. 68	------	------
32	770	96	296	23. 4	172	6. 68	115	2. 26	------	------
33	1, 038	113	279	20. 9	152	4. 88	104	1. 41	------	------
40	839	82	283	17. 2	158	4. 32	104	1. 35	------	------

the empirical tests that are usually employed for measuring the consistency of asphalts. However, in the investigation of these materials, float test determinations were made at 50°, 60°, 70°, 80°, and 90° C. The values obtained at 80° C. were used in the report of Lewis and Welborn [4] for the calculation of the float-test index. The kinematic viscosity determinations made on the selected 50–60 and 85–100 penetration grade asphalts, shown in table 7, were in most cases made at 65°, 75°, and 85° C., but the values at the temperature used for the float-test determinations were interpolated from the curves. These values for the float-test and the kinematic viscosity at the same temperature are shown in table 13. The data in this table have been plotted in figure 6 to logarithmic scales.

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[4] The Physical and Chemical Properties of Petroleum Asphalts of the 50–60 and 85–100 Penetration Grades. Annual Meeting of Association of Asphalt Paving Technologists, Chicago, Ill., Jan. 1940. PUBLIC ROADS, March 1940, vol. 21, No. 1.

In this figure the best straight line has been drawn through the points for each temperature. Some of the points deviate widely from this line, but this is to be expected since the materials tested differed widely in source and inherent characteristics such as specific heat and surface tension. However, in general a good relationship is indicated.

The curve for each test temperature has a different slope. This slope increases for decreasing temperature, the increase becoming progressively greater for each decrement of temperature. It will be noted also that the tests run at the higher temperatures, especially 70° and 80° C., show less deviation from the straight line than those run at 50° and 60° C.

While this general relationship between the float test and kinematic viscosity was found, no definite conversion factor can be determined that would be applicable to all types of asphalt cements. Rhodes, Volkmann, and Barker [3] report this same condition for the float test determination on road tars, even though their work was done on coal tar residues which were perhaps from the same or similar sources and similar types of manufacture.

SUMMARY

The determination of the viscosity of all grades of bituminous materials over a wide range of temperature in absolute units is desirable since it gives a definite basis for comparison of the consistency of all types by a common measure. However, it is recognized that one instrument will not give results over the complete range but measurements by two or three instruments are necessary. These tests have shown that a capillary type instrument, such as the one used in this investigation, will give results from a minimum of approximately 100 centistokes to a maximum of approximately 6,000,000 centistokes. Thus it would be valuable as a unit in any combination of instruments necessary to cover the entire range of consistency of bituminous road materials.

The limitations of the instrument permit viscosities of asphalts of the 50–60 and 85–100 penetration grades to be made only over a temperature range of approximately 45° to 90° C. This range does not permit a study of the consistency of these asphalts at most of the temperatures that are of interest to users of these types of asphaltic materials. In actual service the asphalts are at temperatures from below 0° C. to about 60° C., and they are heated to approximately 135° to 180° C. for use in hot mixed pavements or for application in the construction of penetration macadam. It is not possible to measure the viscosity of these asphalts at atmospheric temperatures with this instrument, although the use of a larger capillary might permit the determinations to be made at lower temperatures than used in this study.

By the use of additional heating coils and a clear oil bath which would not flash at the higher temperature, it might also be possible to extend the range of the instrument to include viscosity measurements at the mixing or application temperatures; but, even with these modifications, the instrument is not readily adapted to cover the consistency measurements of the semisolid asphalts of the grades studied at those temperatures for which such measurements would be of practical significance.

For the liquid asphaltic materials, the instrument can be satisfactorily employed to measure the consistency of all grades at 35° C., the temperature suggested for evaluating the relative consistency of the various grades of road tars. The adoption of a standard method for the determination of the absolute viscosity of all grades of both the liquid asphaltic road materials and road tars at a test temperature, such as 35° C., close to that of normal atmospheric temperature should prove of great value in that it would give a definite knowledge of the consistency of these materials at the time of application and before any appreciable loss of volatile material occurred.

The results obtained with this instrument may be expected to check within ±2 percent or better for these penetration grades of asphalt at the test temperatures used, and this accuracy compares favorably with that usually obtained in ordinary testing of bituminous materials. The instrument gives much closer checks with liquid asphalts of lower viscosities at atmospheric temperatures.

No satisfactory factor for converting the values for float test at various temperatures to kinematic viscosity can be determined that would be applicable to all asphalts.

The data obtained with the instrument can be plotted in a straight line by plotting the double logarithm of the viscosity in centistokes against the logarithm of the absolute temperature, and the slope of this line gives a factor for the viscosity-temperature susceptibility over the range of temperature at which test results can be obtained.

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[3] Consistency Measurements in the Coal Tar Industry, by Rhodes, Volkmann, and Barker. Symposium on Consistency, A. S. T. M., June 1937.

(Continued from p. 126)

IMPORTANCE OF DETERMINING FLEXURAL STRENGTH OF PAVEMENT CONCRETE EMPHASIZED

Figure 5 shows the strength data given in figure 4 plotted against the water-cement ratio instead of the cement factor. Here again the quality of the coarse aggregate is reflected in the results obtained. Note, for instance, that a water-cement ratio of 0.60 was required to develop a modulus of rupture of 550 pounds per square inch with combination 1–10, whereas this strength was obtained with a water-cement ratio of 0.95 in combination 1–1.

Attention is directed to the fact that the curves for combinations 1–1 and 1–10 are straight lines. However, this was not the case for all combinations of materials investigated, many combinations giving curves which had the characteristic shape of the typical water-cement ratio—compressive strength curve. In fact the average curve shown is of this general shape.

The data shown in figures 2 and 3 might have been plotted with the principal relation between water-cement ratio and strength as in figure 5, with additional curves to give the other desired information. The arrangement of data is of course a matter to be decided by circumstances.

Complete compressive strength data are not available for this series of tests because of the fact that companion compressive strength specimens were not made. However, limited data were obtained for the three leanest mixes (4.4, 5.2, and 6.0 sacks of cement per cubic yard) as the result of tests on modified cubes remaining from the flexure tests. These data showed that for combination 1–1 the compressive strengths were 9 percent

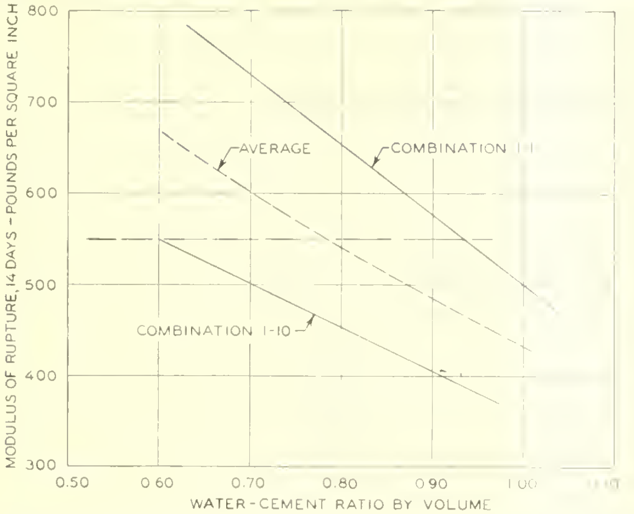

Figure 5.—Relation Between Water-Cement Ratio and Flexural Strength of Concrete. (Average Curve and Maximum Range.)

greater than those for combination 1–10 as compared with the corresponding average flexural strength differential of 32 percent.

Of the group in which sand No. 2 was combined with eight different coarse aggregates, combination 2–13 gave just about as high compressive strength as any combination in the group. However, it was found that, at a cement factor of 6 sacks per cubic yard, the flexural strength of combination 2–13 was the lowest of all, other combinations in the group giving strengths up to 17 percent higher than combination 2–13 (table 4). Many other examples could be given to show that two concretes having the same compressive strength do not necessarily have the same flexural strength, emphasizing the importance of determining the flexural strength characteristics of the constituent materials used in pavement concrete.

It is believed that the data presented are sufficient to indicate that, when pavement concrete is to be made of a variety of combinations of aggregates and a reasonably uniform minimum flexural strength is desired, investigations of the materials should be made along the lines indicated in this report in order to predetermine the flexural strengths to be expected. Compressive strength tests on concrete made with the materials to be used are inadequate, as the ratio of flexural to compressive strength varies over wide limits for different combinations of materials.

INDEX TO PUBLIC ROADS, VOLUME 20, NOW AVAILABLE

The index to PUBLIC ROADS, volume 20, is now available. A chronological list of articles and a list of authors are included with the index. The index will be sent free to subscribers to PUBLIC ROADS requesting it. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

Indexes to volumes 6 to 8 and 10 to 19, inclusive, are also available and will be sent to PUBLIC ROADS subscribers upon request. Indexes to volumes 1 to 5, inclusive, have never been prepared. The supply of the index to volume 9 is exhausted.

REORGANIZATION WITHIN DEPARTMENT OF COMMERCE

Effective August 14, 1940, the name of the Automotive-Aeronautics Trade Division of the Bureau of Foreign and Domestic Commerce, Department of Commerce, has been changed to Motive Products Division. The Railway Equipment Section, formerly a part of the Transportation Division, has been transferred to the Motive Products Division.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF AUGUST 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,464,149	$ 729,767	36.2	$ 4,322,451	$ 2,146,773	164.6	$ 2,089,250	$ 1,040,470	73.6	$ 2,317,772
Arizona	266,375	185,485	12.3	1,196,267	785,707	49.2	351,629	170,320	15.6	1,453,417
Arkansas	1,391,182	617,532	19.4	2,557,404	1,287,053	127.1	468,865	228,867	15.3	904,398
California	1,725,188	924,507	31.1	8,428,219	4,354,359	129.6	2,144,067	1,170,270	49.1	2,314,705
Colorado	688,141	387,836	32.5	2,573,204	1,430,728	130.2	290,719	163,849	93.9	2,375,856
Connecticut	15,000	7,500		2,671,413	1,304,214	21.9	283,541	133,343	4.6	1,060,462
Delaware	52,661	26,330	.9	1,563,392	780,967	16.5	561,996	254,289	15.1	1,048,125
Florida	152,500	76,250	1.1	3,811,683	1,895,563	115.8	786,995	393,498	17.1	2,340,569
Georgia	511,766	255,883	37.1	6,226,919	3,113,460	294.9	4,801,481	2,401,241	174.2	4,888,279
Idaho	411,947	251,915	33.4	1,405,055	825,348	134.8	67,429	41,590	6.7	1,799,201
Illinois	1,071,400	535,700	25.9	9,496,598	4,747,404	197.9	1,710,876	855,438	59.4	3,128,998
Indiana	1,582,010	791,005	40.4	6,663,821	3,325,504	126.0	3,898,700	1,754,500	57.3	951,740
Iowa	1,056,836	443,290	19.9	5,651,428	2,595,462	195.9	2,736,462	1,284,760	93.9	463,961
Kansas	592,276	296,138	38.2	7,159,735	3,594,469	429.9	2,542,552	1,256,328	261.1	3,576,242
Kentucky	810,339	403,613	12.5	4,279,129	2,139,564	95.4	557,235	278,618	69.5	2,991,045
Louisiana	102,203	45,577	.6	11,772,001	2,934,899	36.7	2,520,534	1,248,671	60.0	2,933,530
Maine	322,100	161,050	6.3	1,394,798	697,398	37.6	346,410	173,205	5.9	602,959
Maryland	86,000	43,000	.9	3,846,031	1,914,697	48.1	461,503	228,651	5.3	1,269,091
Massachusetts	537,234	268,191	4.4	3,711,347	1,846,600	30.9	198,728	99,364	.7	2,803,104
Michigan	733,782	366,891	16.9	9,645,810	4,734,804	301.7	3,421,728	1,711,864	90.0	499,128
Minnesota	1,254,476	581,860	79.3	7,550,186	3,764,754	519.0	1,519,753	757,636	95.1	3,134,096
Mississippi	630,400	238,750	30.5	5,923,606	2,642,859	299.0	3,168,760	1,477,830	148.7	1,490,118
Missouri	1,471,813	735,907	69.3	7,015,236	3,324,022	209.1	2,740,970	1,075,023	76.2	4,382,374
Montana	1,772,545	1,005,189	117.3	3,053,507	1,725,537	192.7	711,489	370,833	39.3	3,748,414
Nebraska	1,746,156	873,078	210.4	5,414,542	2,619,255	637.4	1,485,928	742,964	156.7	2,572,484
Nevada	903,337	777,776	49.1	1,667,109	1,446,157	73.9	5,600	4,881		759,186
New Hampshire	363,297	178,779	7.9	1,200,627	591,075	31.6	293,186	141,043	6.0	890,424
New Jersey	1,089,800	544,670	8.9	4,416,148	2,208,074	32.9	152,290	76,145		2,157,070
New Mexico	295,323	182,885	35.6	2,045,879	1,253,915	106.7	569,311	343,181	58.3	1,321,376
New York	1,605,890	798,782	31.2	15,520,362	7,659,680	236.0	3,245,077	1,394,209	34.9	1,769,545
North Carolina	2,018,935	1,008,510	99.8	4,263,331	2,132,197	202.9	1,311,810	655,330	51.3	1,967,891
North Dakota	368,910	197,727	27.4	3,215,338	1,793,666	267.8	3,222,544	1,654,102	274.9	3,247,421
Ohio	885,723	442,862	10.1	11,194,444	5,569,577	111.9	5,015,400	2,507,700	39.1	4,928,659
Oklahoma	709,792	376,887	39.0	3,576,597	1,891,289	117.2	1,431,085	705,629	52.4	4,108,137
Oregon	1,069,898	643,910	95.4	3,054,835	1,819,275	68.2	390,370	235,540	7.3	1,292,674
Pennsylvania	1,111,060	553,190	16.5	10,489,569	5,194,605	114.9	5,368,512	2,659,329	36.6	2,547,885
Rhode Island	203,702	101,635	1.8	1,195,476	596,635	11.9	350,158	174,780	3.8	1,001,817
South Carolina	22,600	11,300	.1	2,704,264	1,304,440	169.6	886,092	427,655	108.4	2,315,306
South Dakota	822,094	472,894	139.8	4,189,890	2,450,540	571.3	1,604,660	990,090	238.2	2,734,371
Tennessee	334,614	167,307	3.5	3,836,352	1,918,176	113.5	1,497,110	748,555	61.6	3,733,138
Texas	2,395,957	1,179,390	193.4	7,486,535	3,711,272	337.2	3,441,092	1,649,645	186.3	6,860,611
Utah	360,045	263,205	32.1	1,182,595	838,293	59.7	465,060	196,660	19.7	924,958
Vermont	134,700	67,350	4.5	1,578,498	787,599	47.2	251,669	125,835	5.1	298,845
Virginia	679,622	293,164	25.2	3,034,347	1,472,840	68.3	1,125,073	560,904	30.2	1,749,171
Washington	848,456	444,394	9.8	3,616,683	1,913,535	82.0	534,391	282,500	9.8	990,478
West Virginia	352,800	175,220	12.6	3,093,970	1,540,696	93.9	1,238,740	618,520	27.2	1,679,556
Wisconsin	1,745,764	857,310	91.1	4,699,141	2,315,800	137.6	489,165	236,430	30.0	3,568,886
Wyoming	995,142	627,108	110.8	852,951	551,358	93.8	593,680	369,207	63.4	892,837
District of Columbia	38,300	19,150	.4	499,942	249,971	4.1	119,500	59,750	1.0	382,475
Hawaii	29,184	11,010		237,910	122,602	2.8	662,741	329,516	11.6	1,496,753
Puerto Rico	20,338	9,980	.2	1,380,599	682,575	25.6	228,404	113,550	.6	764,665
TOTALS	39,853,762	20,658,639	1,923.0	227,567,174	112,547,242	7,724.4	74,360,320	36,574,108	3,042.0	109,484,203

U. S. GOVERNMENT PRINTING OFFICE. 1940

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF AUGUST 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 152,844	$ 76,213	1.4	$ 975,579	$ 486,070	56.3	$ 112,400	$ 56,200	0.3	$ 483,029
Arizona	53.782	17,866	4.7	261,016	188,429	12.8	25,963	18,745	1.0	234,685
Arkansas	331,054	154,443	7.7	271,725	135,651	23.9	114,125	74,526	10.6	70,879
California	341,885	185,938	10.5	624,380	338,814	31.2	137,102	73,776	.5	819,162
Colorado				152,679	63,187	1.2	46,749	26.348	.7	200,688
Connecticut	52,818	25,299	.8	314,408	152,710	3.8	104,991	50,041	1.8	165,763
Delaware	54,721	27,361	7.6	73,439	29,007	5.1	40,371	19,751	2.8	268,125
Florida				351,383	174,893	.4	606,994	278,024	19.8	173,686
Georgia	25,161	12,581	4.6	569,038	269,519	53.5	543,346	271,673	46.2	1,058,954
Idaho				101,331	62,475	15.2	70,844	37,885	7.6	203,486
Illinois	327,500	163,750	20.1	1,833,485	899,568	67.4	324,400	162,200	14.7	304,077
Indiana	227,670	113,835	16.8	314,102	155,812	17.9	91,936	45,900	4.4	916,997
Iowa	763,200	362,275	141.3	1,145,674	546,216	223.2	840,411	396,280	241.1	235,697
Kansas	202,136	101,068	1.9	741,979	373,898	52.5	287,357	143,678	46.9	1,212,201
Kentucky	453,306	127,985	25.9	553,896	202,045	43.3	294,932	80,800	15.0	332,278
Louisiana	41,637	20,818	3.7	256,292	128,091	22.0				458,796
Maine	72,650	35,536	4.5	189,474	91,428	10.8	71,200	33,500	3.2	7,907
Maryland				177,996	88,998	8.2				427,378
Massachusetts	110,536	54.886	2.7	523,730	259,616	10.7				497,383
Michigan	72,760	36,380	12.2	1,524,117	765,448	114.2	155,760	77,880	15.3	578,361
Minnesota	125,732	59,354	6.6	693,263	345,915	104.3	333,167	166,583	68.8	1,026,170
Mississippi	146,300	73,150	10.6	526,814	257,907	12.0	593,020	284,975	39.2	421,072
Missouri	177,806	88,903	24.8	515,128	254,153	49.7	362,668	114,512	46.2	703,803
Montana	429,411	242,892	61.4	249,798	140,960	19.2	29,357	16,566	2.3	655,794
Nebraska	241,075	120,379	43.8	671,543	340,348	82.7	97,331	43,569	23.4	287,707
Nevada	84,598	73,231	15.5	168,076	140,559	27.1	30,543	23,629	3.9	89,390
New Hampshire	48,387	23,241	2.2	37,254	16,906	.1	71,651	35,488	1.2	162,156
New Jersey	254,560	127,280	10.5	329,030	164,435	9.1	61,333	30,660	2.4	499,449
New Mexico	101,564	63,386	13.1	396,987	244,848	14.7	247,335	84,392	11.9	101,030
New York	688,360	344,180	24.8	2,213,924	1,056,282	59.9	350,540	136,870	14.4	171,955
North Carolina	232,540	116,270	21.1	867,213	434,573	76.6				327,757
North Dakota	39,433	21,136	.3	145,151	79,399	1.2	27,520	14,750	2.4	1,014,688
Ohio	244,707	122,288	10.4	2,329,350	1,171,450	82.6	807,841	402,550	17.9	791,442
Oklahoma	450,988	239,251	31.8	216,365	114,370	16.1	267,180	116,876	12.4	929,254
Oregon	189,797	96,650	29.8	304,017	137,910	31.5	135,049	79,280	15.0	241,854
Pennsylvania	449,502	218,646	19.4	1,682,197	840,190	45.8	195,984	97,992	4.7	235,836
Rhode Island	2,716	1,329		227,397	113,672	3.7				95,049
South Carolina	41,500	18,100	3.8	616,040	235,716	57.2	184,830	74.850	56.4	225,295
South Dakota				3,624	3,624					1,280,913
Tennessee	91,090	45,545	7.5	62,560	31,280	3.4	186,188	93,094	7.2	909,350
Texas	738,992	363,625	106.2	690,048	330,210	80.0	334,950	160,725	37.3	982,372
Utah				195,647	120,000	16.1	81,845	58,600	15.6	118,707
Vermont	139,900	48,305	5.4	366,408	103,750	14.0				22,500
Virginia	168,540	80,212	9.0	454,710	207,319	24.7	123,866	51,685	1.6	284,724
Washington	94,191	49,488	2.8	528,552	280,039	28.1	156,374	80,000	17.3	184,084
West Virginia	123,800	61,375	7.5	186,819	93,409	9.2	79,808	39,904	4.4	446,189
Wisconsin	180,211	89,800	2.2	525,609	262,710	16.2	221,661	110,800	11.5	653,577
Wyoming	261,777	165,255	29.8	183,125	103,159	13.0	58,269	37,245	4.0	113,561
District of Columbia				43,684	21,342	.6	92,200	37,500	.8	28,150
Hawaii	164,258	82,085	6.4	114,489	57,793	2.2				158,778
Puerto Rico				302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	9,195,395	4,551,590	773.1	26,802,770	13,263,743	1,688.6	9,054,579	4,267,442	856.2	21,892,546

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF AUGUST 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama						$ 722,004	$ 701,927	6		1	$ 16,800	$ 16,800			5	$ 943,965
Arizona	$ 354,097	$ 319,636	4			9,473	9,473				6,006	6,006			2	276,646
Arkansas	10,230	10,230			4	1,154,833	1,150,686	8	1		232,393	232,180	5		12	411,345
California	181,075	181,075	2		3	886,146	707,665	3	1		18,276	18,276			9	1,696,719
Colorado						277,100	277,100	1			9,522	9,522			1	918,723
Connecticut	153,792	148,343		1		631,483	625,932	7								451,839
Delaware	39,821	39,821			8	106,826	106,826	1		3	50,450	50,450	4		6	467,605
Florida	114,447	109,949	1		1	88,900	88,900	1			168,970	168,445	2		16	1,287,491
Georgia	81,253	81,253	2		7	733,165	733,165	9	4	1	657,241	657,241	4	4	9	1,791,106
Idaho	104,842	104,237	1			102,897	100,072	4			85,358	85,358			33	424,756
Illinois	109,232	107,621	1		4	2,858,728	2,597,922	11	2	42	260,966	213,247	1		73	1,859,346
Indiana	77,455	77,455		1	17	1,019,801	1,019,801	7	1	44	112,783	112,783			35	1,006,040
Iowa	88,665	83,100			23	313,303	239,822	4		27	249,577	235,392	2		47	1,210,949
Kansas	7,242	7,241			3	878,803	878,325	12		3	270,788	270,788	3	2	16	1,095,945
Kentucky	157,925	157,925	4		9	1,007,185	1,007,185	9			44,325	44,325	2		3	571,318
Louisiana						440,618	387,123	3			627,884	570,154	11			801,459
Maine	73,922	73,922		1	4	97,898	97,898	1	1		132,585	132,585	1		2	251,016
Maryland	4,600	4,600			1	636,709	604,916	3	2	1	27,800	27,800			8	770,425
Massachusetts	15,710	15,710				336,591	326,168	1								2,025,042
Michigan	193,014	193,014	1	1	7	1,909,627	1,909,627	8	4	5	158,572	158,572			24	908,161
Minnesota	507,767	499,910	4	1		1,458,599	1,458,599	12	3	9	220,385	220,385		2	9	1,055,027
Mississippi						506,594	506,594	6	1		91,300	91,300	3		1	828,349
Missouri	150,832	150,832		2		1,033,074	1,033,074	6	2		1,725,711	1,231,145	4	2	1	1,090,903
Montana	281,054	281,054	5		2	154,873	85,232				99,948	96,682	1		2	354,768
Nebraska	188,369	188,369		1	3	561,934	561,934	6	1	5	194,103	194,103	4		2	563,847
Nevada						115,732	115,732	2		5	42,138	42,138			20	115,071
New Hampshire	100,989	100,953	3			80,259	80,259	1			3,324	3,324			1	390,249
New Jersey						772,219	772,219	5			69,720	69,720		1	1	1,214,345
New Mexico	20,902	20,902			6	230,045	230,045	2			102,612	97,850	2			548,809
New York	307,110	306,000	2	1		3,766,739	3,699,173	10	17		461,795	457,195		3	8	2,712,937
North Carolina	104,415	104,415	1	1	5	980,368	980,368	13	2	8	111,740	111,440		2	17	858,246
North Dakota	314,410	314,410	4		3	210,780	210,780	3		11	143,020	119,223	1		9	741,485
Ohio	276,468	254,706	3		1	2,518,025	2,468,835	11	3	10	473,067	448,075	2		5	2,614,346
Oklahoma	132,719	131,692	2		7	632,962	632,962	9	1	9	284,817	279,431	2		49	1,881,590
Oregon						462,848	315,518	4								363,725
Pennsylvania	34,709	34,709		1		1,936,810	1,927,334	17	1		977,336	977,131	7			3,949,718
Rhode Island	3,831	3,750				191,039	191,039		1							95,913
South Carolina	79,742	79,742	2	1	1	389,449	389,449	3	2	9	145,000	144,597			24	974,116
South Dakota						228,022	227,162	5	1	4	227,400	211,450	5			1,102,307
Tennessee	160,052	160,052	1	1		92,736	92,736	1	1		124,741	124,741		1		1,749,947
Texas	291,059	290,200	4		3	2,006,677	1,992,363	16	1	4	75,150	75,150	1		4	2,030,627
Utah	7,420	7,420			2	42,751	42,751			11	54,011	54,011			20	297,988
Vermont	2,969	2,969			1	209,428	209,428		2	1	34,546	34,546			7	223,562
Virginia	17,400	17,400	1		3	280,068	278,402	3	2		272,447	272,447	1	2	8	940,287
Washington	138,380	138,380	1	1	2	220,472	218,972	3	1	1	78,830	78,830	1		6	513,498
West Virginia						229,982	229,982	2			5,400	5,400			2	1,144,537
Wisconsin	337,377	333,340	4	1	7	936,427	896,342	5	3	3	13,998	13,998			4	1,371,022
Wyoming						540,923	540,923	5								192,955
District of Columbia						11,061	11,061				48,000	48,000	1			150,010
Hawaii						194,036	194,029	2			9,494	9,494			2	290,147
Puerto Rico						584,007	579,336	11								414,857
TOTALS	5,225,296	5,136,337	53	15	137	35,791,029	34,743,166	262	61	217	9,220,329	8,521,730	70	19	503	49,345,084

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH *Editor*

Volume 21, No. 8 October 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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STATE OF IMPROVEMENT OF RURAL ROADS IN RELATION TO TRAFFIC AND DWELLINGS SERVED

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by **JOHN T. LYNCH**, Highway Engineer-Economist and **THOMAS B. DIMMICK**, Associate Highway Engineer-Economist

AN IMPORTANT PART of the State-wide highway planning surveys has been the determination of the state of improvement of rural roads in relation to the volume of travel and to the location of rural dwellings. Such a determination is basic in any appraisal of highway service in relation to needs, and in the setting up of long-range construction programs and the making of financial provisions for them.

The summarization of the data has not yet been completed in all States, but the work is sufficiently far advanced to permit the release of preliminary figures that give a reasonably accurate national picture of our highway facilities, showing the extent to which traffic and rural dwellings are served by roads having surfaces of different types. A complete appraisal of the adequacy of highways for traffic needs would necessarily take into consideration surface width, alinement, grades, and sight distances, as well as surface type. The clearances, strength, and condition of bridges, and the hazards and delays at railroad grade crossings, should also be considered. Information concerning all of these factors has been obtained in the highway planning surveys, but is not yet ready for presentation on a national basis. This article deals only with the service rendered by roads of different surface types in those States for which the information is now available.

Road construction practices vary in different sections of the country because of differences in climatic conditions, subsoil conditions, and character of available local materials as well as differences in wealth and in public policy as carried out by the State and local highway organizations. There are, then, no standards of adequacy that are generally applicable throughout the country. In one section, climatic conditions may be such that unsurfaced roads will satisfactorily serve much higher volumes of traffic than in another section. Subsoil conditions in one locality may require a higher type of surface for traffic of a given volume and weight composition than that required in another locality. Differences in available funds and mileages of roads needing improvement have caused States to adopt different standards of improvement for roads of equal traffic importance. These facts should be borne in mind in appraising the relative degrees of surface improvement shown by the tabulations in this article.

ROADS CLASSIFIED INTO FIVE GROUPS ACCORDING TO SURFACE TYPE

Because of differences in construction practices and in current terminology, considerable difficulty was experienced in classifying surface types on a comparable basis in all States. Although every effort was made to attain uniformity, minor differences in classification undoubtedly exist. In preliminary tabulations 12 classifications were used but these are combined into five groups in the accompanying tables. These five general surface type groups are defined in commonly used terms, without attempting to make precise distinctions. The terms used to describe the individual types composing each group are not, in all cases, mutually exclusive, as two or more types may be nearly identical. The distinctions between the general surface type groups, however, are reasonably definite in all cases.

1. *Pavement* includes concrete, brick, stone block, wood block, asphalt block, sheet asphalt, rock asphalt, bituminous concrete, and bituminous penetration.
2. *Other dustless surface* includes plant mix without precise control, road mix or mixed-in-place, and bituminous surface treated gravel or stone.
3. *Nondustless surface* includes plain macadam, gravel, traffic-bound crushed stone, slag, chert, caliche, iron ore, chats, sand-clay, and topsoil.
4. *Graded and drained* includes roads of natural earth, alined and graded to permit reasonably convenient use by motor vehicles, with drainage systems sufficient to prevent serious impairment of the road by surface water.
5. *Unimproved* includes primitive roads and trails usable by four-wheel vehicles and also earth roads on which some blading may have been done but which do not conform in respect to alinement, grade, and drainage to the requirement for a graded and drained road.

Roads with surfaces falling in the first three groups are called surfaced roads; roads falling in the last two groups are called unsurfaced roads.

The initial road inventory and traffic surveys were started in a few States as early as the fall of 1935 and completed toward the end of 1936. The work was started later in most States and, in a few, only very recently. The effective dates of figures in the accompanying tables vary from 1936 to 1939 and the period covered by the traffic survey does not, in all cases, coincide with that covered by the road inventory. Under the continuing survey an effort is being made to keep traffic and road inventory information current so that it should soon be possible to present up-to-date tabulations of both road inventory and traffic data for a specific year.

In interpreting the figures given in the tables and text of this report, it should be borne in mind that there are some differences in effective dates and that the data apply to different periods from 1936 to 1939. There have undoubtedly been changes since the effective dates of the data, because of the normal construction programs of Federal, State, and local agencies, and the surfacing of considerable mileages with the assistance of the Work Projects Administration in a number of States. On the whole, however, it is believed that changes have not been sufficiently large, relatively, to invalidate the general picture presented.

The total mileage of rural roads in the United States is estimated to be approximately 2,960,000 miles. Of this, approximately 1,200,000 miles, or about 40 percent, have surfaces permitting travel in all seasons of the year. Complete information on road mileage and surface type as of various dates from August 1936 to December 1939 is available for 34 States. The total road mileage reported by these States was 2,219,723 miles, of which 840,129 miles, or 37.8 percent, were surfaced. Table 1 shows the mileage of each general surface type and the percentage of the total mileage for each of the 34 States.

The percentage of the total mileage that was surfaced ranged from 85.4 percent in Ohio, to 13.5 percent in Nevada. It will be noted that most of the surfaced mileage in Ohio consisted of gravel and other nondustless types, only 25.1 percent of the total mileage in this State having dustless surfaces. In New Jersey, on the other hand, in which 62.7 percent of the total mileage was surfaced, 49.4 percent had dustless surfaces. Nine States had less than 20 percent of their total road mileages surfaced. Figure 1 shows the percentage of the total mileage surfaced, the percentage with a dustless surface, and the percentage paved, for each of the 34 States, arranged in descending order of the percentage surfaced.

TRAFFIC ON PAVED ROADS 12 TIMES THE AVERAGE FOR ALL ROADS

Naturally, travel is generally heavier on roads having high type surfaces than it is on low type surfaces or on unsurfaced roads. In the first place, high traffic volume, actual or potential, was generally the cause of the construction of the higher type surfaces. In addition, traffic tends to gravitate to the more highly improved roads.

Combined traffic and road inventory information is available for 24 States. In these States the average daily traffic was 1,232 vehicles for pavements, 413 vehicles for other dustless surfaces, 77 vehicles for nondustless surfaces, 22 vehicles for graded and drained roads, and 13 vehicles for unimproved roads. The average for all types of road was 104 vehicles. Table 2 shows average daily traffic on each general surface

TABLE 2.—*Average daily traffic on rural roads of each general surface type in each of 24 States*

State	Pavement	Other dustless surface	Non-dustless surface	Graded and drained	Unimproved	All roads
Arizona	946	522	83	50	13	84
California	1,485	266	83	40	17	236
Colorado	2,096	575	84	32	10	59
Florida	988	353	47	27	10	173
Idaho	1,896	551	79	25	12	70
Indiana	1,370	370	53	10	6	165
Iowa	1,014	386	87	25	17	94
Kansas	1,239	590	98	54	16	64
Louisiana	836	789	96	12	6	113
Maryland	1,285	565	65	35	21	338
Michigan	1,525	324	84	22	10	160
Missouri	1,303	377	60	14	4	76
Montana	505	293	49	26	9	38
Nevada	1,535	233	38	22	5	33
North Dakota	1,641	484	67	11	2	17
Ohio	1,146	332	64	20	18	204
Oklahoma	1,356	638	153	24	6	78
Oregon	1,084	398	60	9	3	89
South Dakota	1,065	517	78	12	1	27
Texas	1,199	641	95	78	25	122
Utah	1,418	416	57	17	5	72
Vermont	916	535	68	11	3	111
West Virginia	992	485	105	52	26	147
Wyoming	742	357	88	27	12	56
Average	1,232	413	77	22	13	104

TABLE 1.—*Mileage of rural roads of each general surface type and percentage of total mileage of rural roads in each of 34 States*

State	Date of inventory	Pavement		Other dustless surface		Nondustless surface		Total surfaced		Graded and drained		Total improved		Unimproved [1]		Total	
		Miles	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*
Arizona	Dec. 1937	564.5	2.0	2,174.4	7.9	2,317.8	8.4	5,056.7	18.3	4,218.8	15.3	9,275.5	33.6	18,271.7	66.4	27 547.2	100.0
Arkansas	Jan. 1937	1,595.4	2.9	624.0	1.2	12,528.6	23.1	14,748.0	27.2	11,648.9	21.5	26,396.9	48.7	27,889.6	51.3	54,286.5	100.0
California	Dec. 1937	10,114.7	10.2	22,183.8	22.3	[2] 20,333.0	20.4	52,631.5	52.9	3,810.4	3.8	56,441.9	56.7	[3] 43,118.6	43.3	99,560.5	100.0
Colorado	Aug. 1939	493.4	.7	3.218.6	4.3	9,307.9	12.3	13,019.9	17.3	6,658.2	8.8	19,678.1	26.1	55,576.7	73.9	75,254.8	100.0
Florida	Dec. 1936	1,835.7	6.4	7,599.5	26.3	2,435.9	8.4	11,871.1	41.1	13,828.6	47.8	25,699 7	88.9	3,212.1	11.1	28,911.8	100.0
Idaho	Dec. 1936	171 9	.5	1,940.2	5.8	7,792.1	23.2	9,904.2	29.5	4,484.1	13.4	14,388.3	42.9	19,152.2	57.1	33,540.5	100.0
Illinois	Jan. 1937	11,390.9	11.1	678.8	.7	47,899.5	46.6	59,969.2	58.4	37,747.5	36.8	97,716.7	95.2	4,966.9	4.8	102,683.6	100.0
Indiana	Jan. 1937	5,452.4	7.1	6,390.8	8.3	49.872.6	65.1	61,715.8	80.5	5,690.4	7.4	67,406.2	87.9	9.283.9	12.1	76,690.1	100.0
Iowa	Dec. 1937	4,843.5	4.8	558.9	.5	35,077.3	33.8	40,479.7	39.1	40,794.9	40.4	81,274.6	79.5	[4] 20,624.3	20.5	101,898.9	100.0
Kansas	Dec. 1936	1,809.8	1.4	3,341.7	2.6	24,756.3	19.3	29,907.8	23.3	2,004.6	1.6	31,912.4	24.9	96,285.4	75.1	128,197.8	190.0
Kentucky	July 1938	1,910.2	3.4	5,758.0	10.2	18,741.6	34.0	26,409.8	47.6	4.033.3	7.2	30,443.1	54.8	25,837.8	45.2	56,280.9	100.0
Louisiana	Dec. 1937	3,405.4	8.8	29.9	.1	13,198.0	34.1	16,633.3	43.0	15,701.0	40.5	32,334.3	83.5	6,387.2	16.5	38,721.5	100.0
Maryland	Jan. 1938	2,447.1	15.3	3,227.7	20.2	3,243.7	20.3	8,918.5	55.8	4,684.4	29.3	13,602.9	85.1	2,380.2	14.9	15,983.1	100 0
Michigan	Dec. 1936	5,464.5	5.9	4,342.9	4.7	48,991.9	53.1	58,799.3	63.7	21,837.3	23.8	80,636.6	87.5	11.509.5	12.5	92,146.1	100.0
Missouri	Dec. 1936	3,998.0	3.4	3,117.5	2.7	29,406.1	25.2	36,521.6	31.3	60,456.3	51.8	96,977.9	83.1	19,716.0	16.9	116,693.9	100.0
Montana	July 1937	1,466.4	2.2	2,965.7	4.5	6,248.7	9.5	10,680.8	16.2	3,389.9	5.2	14,070.7	21.4	51,659.4	78.6	65,730.1	100.0
Nebraska	Dec. 1936	1,034.2	1.0	861.7	.9	16,333.8	16.3	18,229.7	18.2	9,315.3	9.3	27,545.0	27.5	72,770 0	72.5	100,315.0	100.0
Nevada	Dec. 1937	51.5	.2	2,338.6	10.0	761.2	3.3	3,151.3	13.5	865.0	3.7	4,016.3	17.2	19.257.6	82.8	23,273.9	100.0
New Hampshire	Dec. 1937	413.6	3.4	3,404.7	28.0	3,943.0	32.5	7,761.3	63.9	2,725.2	22.4	10,486.5	86.3	1,656.4	13.7	12,142.9	100.0
New Jersey	July 1939	4,852.0	26.2	4,292.2	23.2	2,475.3	13.3	11,619.5	62.7	342.2	1.9	11,961.7	64.6	6,542.1	35.4	18,503.8	100.0
North Carolina	June 1938	4,975.0	8.6	4,419.0	7.6	20,759.2	35 9	30,153.2	52.1	23,490.9	40.6	53,644.1	92.7	4,162.7	7.3	57,806.8	100.0
North Dakota	Jan. 1938	24.5	([5])	689.1	.6	16,702.2	15.2	17,415.8	15.8	19,261.5	17.5	36,677.3	33.3	73,376.2	66.7	110,053.5	100.0
Ohio	Jan. 1937	8,059.9	9.8	12,653.4	15.3	49,700.6	60.3	70,413.9	85.4	7,012.7	8.5	77,426.6	93.9	5,022.4	6.1	82,449.0	100.0
Oklahoma	Jan. 1937	2.696.3	2.7	1.369.4	1.3	11,161.8	11.0	15,227.5	15.0	65,648.5	64.7	80,876.0	79.7	20,529.3	20.3	101,405.3	100.0
Oregon	Sept. 1936	1.799.7	3.9	2,907.9	6.2	14,599.3	31.2	19,306.9	41.3	15,118.7	32.3	34,425.6	73.6	12,340.8	26.4	46,766.4	100.0
South Carolina	Dec. 1938	2,326.2	5.4	3,854.0	9.0	6,830.3	16.0	13,010.5	30.4	11,089.2	25.9	24,099.7	56.3	18.676.1	43.7	42,775.8	100.0
South Dakota	Jan. 1937	228.7	.2	895.2	.9	18,629.1	18.4	19,753.0	19.5	40,920.3	40.5	60,673.3	60.0	40,471.8	40.0	101,145.1	100.0
Texas	Sept. 1937	6,579.8	3.5	12,485.9	6.7	25,170.8	13.6	44,236.5	23.8	16,058.7	8.6	60,295.2	32.4	125,565.5	67.6	185,860.7	100.0
Utah	Jan. 1937	364.9	1.7	1,481.6	6.9	5,357.6	24.9	7,204.1	33.5	3,640.6	17.0	10,844.7	50.5	10,633.0	49.5	21,477.7	100.0
Vermont	Dec. 1938	895.2	6.6	438.0	3.3	5,659.9	42.0	6,993.1	51.9	4,384.0	32.6	11,377.1	84.5	2,092.9	15.5	13,470.0	100.0
Washington	Dec. 1939	1,972.7	4.0	2,997.2	6.2	20.034.7	41.2	25,004.6	51.4	13.258.7	27.3	38,263.3	78.7	10,377.9	21.3	48,641.2	100.0
West Virginia	Jan. 1937	2,437.4	7.5	2,053.8	6.3	6,134.2	18.8	10.625.4	32.6	6,393.0	19.6	17,018.4	52.2	15,571.7	47.8	32,590.1	100.0
Wisconsin	Dec. 1936	4,927.4	6.0	5,751.6	7.0	48,035.8	58.4	58,714.8	71.4	19,624.7	23.9	78,339.5	95.3	3,853.9	4.7	82,193.4	100.0
Wyoming	Dec. 1937	14.8	.1	2,826.1	11.4	1,200.1	4.8	4,041.0	16.3	1,185.4	4.8	5,226.4	21.1	19,498.4	78.9	24,724.8	100.0
Total		100,617.6	4.5	133,871.8	6.0	605,639.9	27.3	840,129.3	37.8	501,323.2	22.6	1,341,452.5	60.4	878,270.2	39.6	[6] 2,219,722.7	100.0

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[1] Includes trails.
[2] Includes 6,727.7 miles of oiled earth.
[3] Includes 22.4 miles, surface type unreported.
[4] Includes 274.1 miles, surface type unreported.
[5] Less than 0.05 percent.
[6] States not listed estimate their total mileage at 739,154 miles, giving total for the country as 2,958,877 miles of rural highway.

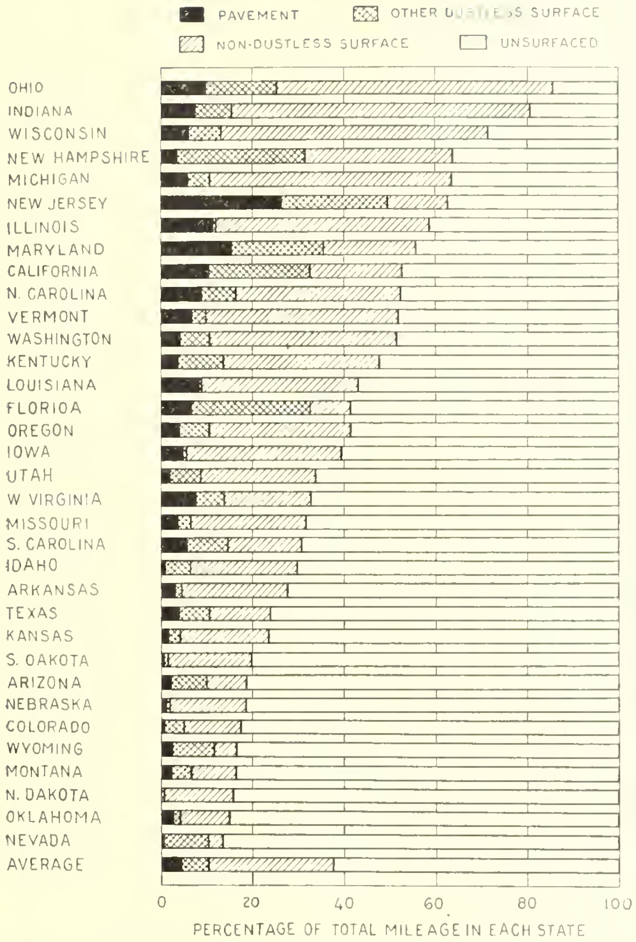

Figure 1.—Percentage of Total Rural Road Mileage in Each of 34 States With Surfaces of Each General Type.

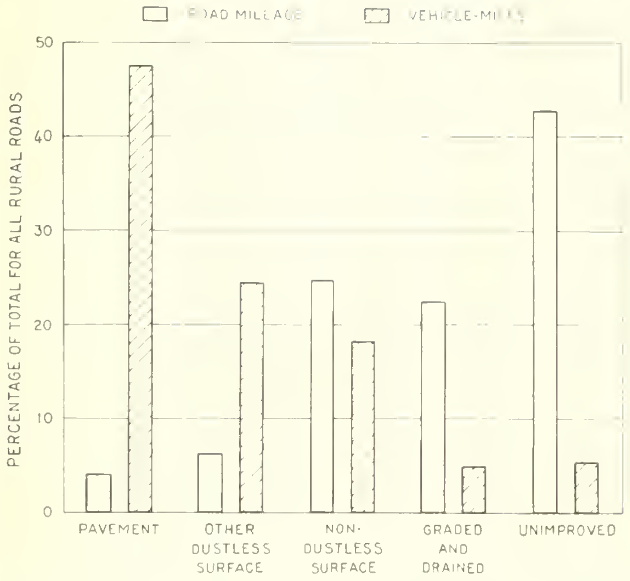

Figure 2.—Percentage of Rural Road Mileage Compared to Percentage of Vechicle-Miles of Travel for Each General Surface Type in 24 States.
States Included Are: Arizona, California, Colorado, Florida, Idaho, Indiana, Iowa, Kansas, Louisiana, Maryland, Michigan, Missouri, Montana, Nevada, North Dakota, Ohio, Oklahoma, Oregon, South Dakota, Texas, Utah, Vermont, West Virginia, and Wyoming.

type for each of the 24 States. Maryland had an average daily traffic on all roads of 338 vehicles, the highest of any of the States listed; while North Dakota had an average daily traffic on all roads of but 17 vehicles, the lowest of any of the States listed. In all of the States the average daily traffic was successively higher for each successively higher type surface.

Though the average daily traffic on paved roads is much greater than that on roads with lower type surfaces and on unsufraced roads, it does not follow that all of the paved mileage is more heavily traveled than any of the unpaved mileage. Table 3 shows the mileage and percentage of each general surface type in different average daily traffic volume groups in 23 of the 24 States listed in table 2. Data for Indiana are not included in table 3 because tabulations showing mileages of each surface type in traffic volume groups in that State were not available.

Table 3 shows that, in the 23 States, almost 5,000 miles of paved roads carry less than 100 vehicles per day while more than 10,000 miles of unimproved roads and nearly 12,000 miles of graded and drained roads carry higher volumes of traffic. This may reflect on the judgment exercised, in some cases, in selecting roads for improvement, or may indicate that considerations other than traffic importance influenced the selection. There are valid reasons, however, why such conditions should exist with respect to a portion of the mileage. Some of the paved mileage was lightly traveled during the period of the survey because of construction work on other parts of the road, temporarily diverting to other roads part of the normal traffic. Likewise, some of the paved sections connected with sections that are not yet improved and large traffic increases may be expected when the road is completely improved from one population center to another. On the other hand, some of the heavily traveled graded and drained mileage may have been temporarily in that status, during the course of stage construction. The true extent of overdevelopment or underdevelopment of highway facilities cannot be determined from summary tables of this kind, but can be determined only by a study of each individual road section, taking into consideration all of the pertinent circumstances. Such an approach is being used in a number of States in setting up programs for future improvement and in determining priorities for improvement.

PAVED ROADS CARRIED NEARLY HALF OF THE TOTAL TRAVEL ON ALL ROADS

In the 24 States listed in table 2, the average daily travel on all rural roads amounted to about 169,523,000 vehicle-miles. This is equivalent to 1,695,230 vehicles traveling an average of 100 miles each per day, or to 6,780,920 vehicles traveling an average of 25 miles each per day. About 152,309,000 vehicle-miles, or 90 percent of the total travel, was on surfaced roads, and only about 17,214,000 vehicle-miles on unsurfaced roads. The vehicle-mileages reported for each general surface type by the 24 States were as follows:

	Vehicle-miles
Pavement	80, 262, 267
Other dustless surface	40, 791, 208
Nondustless surface	31, 255, 837
Graded and drained	8, 258, 911
Unimproved	8, 955, 018
Total	169, 523, 241

Table 4 shows for each of the 24 States, the percentage distribution of the total vehicle-mileage of travel by surface types in comparison with the percentage distribution of the total road mileage by surface types. It shows that paved roads, constituting only 4.0 percent of the total rural road mileage in these States, carried 47.3 percent of the total vehicles-mileage of travel. Surfaced roads of all types, constituting 34.9 percent of the total rural road mileage, carried 89.8 percent of the total vehicle-mileage. Unimproved roads constituted 42.7 percent of the total road mileage but

TABLE 3.—*Mileage of rural roads and percentage of total rural mileage of each general surface type in different average daily traffic volume groups in 23 States* [1]

Average daily traffic	Pavement		Other dustless surface		Nondustless surface		Graded and drained		Unimproved		Total	
	Miles	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*	*Miles*	*Percent*
0-24	1, 494. 4	2. 51	9, 562. 4	10. 11	119, 650, 2	33. 64	277, 841. 9	76. 94	606, 179. 6	87. 60	1, 014, 728. 5	64. 92
25-49	2, 189. 6	3. 67	4, 372. 3	4. 62	75, 522. 6	21. 24	48, 730. 1	13. 50	51, 418. 5	7. 43	182, 233. 1	11. 66
50-99	1, 285. 1	2. 16	9, 451. 6	9. 99	74, 497. 2	20. 95	22, 526. 4	6. 24	24, 099. 8	3. 48	131, 860. 1	8. 44
100-199	3, 135. 3	5. 26	15, 061. 4	15. 93	53, 045. 8	14. 92	8, 171. 9	2. 26	7, 749. 3	1. 12	87, 163. 7	5. 58
200-299	3, 111. 4	5. 22	12, 258. 6	12. 96	17, 217. 5	4. 84	2. 012. 5	. 56	1, 688. 6	. 24	36, 288. 6	2. 32
300-399	3, 135. 3	5. 26	9, 506. 1	10. 05	7, 783. 4	2. 19	723. 4	. 20	477. 8	. 07	21, 626. 0	1. 38
400-499	3, 244. 7	5. 44	7, 993. 0	8. 45	3, 604. 6	1. 01	395. 8	. 11	173. 5	. 03	15, 411. 6	. 99
500-599	3, 410. 5	5. 72	6, 009. 4	6. 35	1, 900. 6	. 54	235. 5	. 07	119. 4	. 02	11, 675. 4	. 75
600-699	3, 434. 1	5. 76	4, 374 6	4. 63	858. 3	. 24	109. 4	. 03	62. 3	. 01	8, 838. 7	. 57
700-799	3, 493. 0	5. 86	3, 702. 0	3. 92	575. 4	. 16	77. 1	. 02	28. 3	(2)	7, 875. 8	. 50
800-899	3, 092. 7	5. 19	2, 572. 1	2. 72	334. 7	. 09	57. 7	. 02	30. 8	(2)	6, 088. 0	. 39
900-999	2, 682. 2	4. 50	1, 726 3	1. 83	163. 8	. 05	39. 2	. 01	14. 2	(2)	4, 625. 7	. 30
1,000-1,249	5, 980. 0	10. 03	2, 966. 5	3. 14	229. 7	. 06	40. 3	. 01	16. 0	(2)	9, 232. 5	. 59
1,250-1,499	4, 474. 5	7. 50	1, 940. 8	2. 05	127. 1	. 04	21. 1	. 01	8. 7	(2)	6, 572. 2	. 42
1,500-1,999	5, 941. 8	9. 96	1, 730. 5	1. 83	63. 1	. 02	34. 9	. 01	15. 9	(2)	7, 786. 2	. 50
2,000-2,999	5, 243. 6	8. 79	928. 4	. 98	51. 2	. 01	10. 9	(2)	8. 2	(2)	6, 242. 3	. 40
3,000-3,999	1, 862. 9	3. 12	210. 9	. 25	17. 9	(2)	24. 7	. 01			2, 146. 4	. 14
4,000-4,999	1, 008. 7	1. 69	111. 3	. 12	4. 6	(2)					1, 124. 6	. 07
5,000-5,999	664. 0	1. 11	24. 8	. 03	11. 4	(2)					700. 2	. 04
6,000-6,999	282. 2	. 47	21. 8	. 02							304 0	. 02
7,000-7,999	208. 1	. 35	4. 9	. 01							213. 0	. 01
8,000-8,999	83. 3	. 14	1. 0	(2)							84. 3	. 01
9,000-9,999	41. 1	. 07	. 3	(2)							44 4	(2)
10,000-12,499	66. 9	. 11	3. 1	(2)							70. 0	(2)
12,500-14,999	29. 0	. 05	5. 2	. 01							34. 2	(2)
15,000-19,999	30. 7	. 05									30. 7	(2)
20,000-24,999	3. 5	. 01									3. 5	(2)
25,000-29,999	2 3	(2)									2. 3	(2)
30,000 and over												
Total	59. 633. 9	100. 00	94, 569. 3	100. 00	355, 659. 1	100. 00	361, 052. 8	100. 00	692, 090. 9	100. 00	1, 563, 006. 0	100. 00
Percentage of total mileage	3. 82		6. 05		22. 75		23. 10		44. 28		100. 00	

[1] The States included are: Arizona, California, Colorado, Florida, Idaho, Iowa, Kansas, Louisiana, Maryland, Michigan, Missouri, Montana, Nevada, North Dakota, Ohio, Oklahoma, Oregon, South Dakota, Texas, Utah, Vermont, West Virginia, and Wyoming

[2] Less than 0.005 percent.

TABLE 4.—*Mileage of rural roads of each general surface type and vehicle-miles of travel on each, expressed as percentages of totals for all types in each of 24 States*

State	Pavement		Other dustless surface		Nondustless surface		Total surfaced		Graded and drained		Total improved		Unimproved		Total	
	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles	Percentage of mileage	Percentage of vehicle-miles
Arizona	2. 0	23. 1	7. 9	49. 0	8. 4	8. 3	18. 3	80. 4	15. 3	9. 1	33. 6	89. 5	66. 4	10. 5	100. 0	100. 0
California	10. 2	63. 9	22. 3	25. 1	20. 4	7. 2	52. 9	96. 2	3. 8	. 6	56. 7	96. 8	43. 3	3. 2	100. 0	100. 0
Colorado	. 7	23. 1	4. 3	41. 3	12. 3	17. 9	17. 3	82. 3	8. 8	4. 8	26. 1	87. 1	73. 9	12. 9	100. 0	100. 0
Florida	6. 4	36. 2	26. 3	53. 5	8. 1	2. 3	41. 1	92. 0	47. 8	7. 4	88. 9	99. 4	11. 1	. 6	100. 0	100. 0
Idaho	. 5	13. 9	5. 8	45. 6	23. 2	26. 3	29. 5	85. 8	13. 4	4. 7	42. 9	90. 5	57. 1	9. 5	100. 0	100. 0
Indiana	7. 1	59. 3	8. 3	18. 7	65. 1	21. 2	80. 5	99. 2	7. 4	. 4	87. 9	99. 6	12. 1	. 4	100. 0	100. 0
Iowa	4. 8	51. 4	. 5	2. 2	33. 8	32. 1	39. 1	85. 7	40. 4	10. 6	79. 5	96. 3	20. 5	3. 7	100. 0	100. 0
Kansas	1. 4	27. 2	2. 6	23. 9	19. 3	29. 4	23. 3	80. 5	1. 6	1. 3	24. 9	81. 8	75. 1	18. 2	100. 0	100. 0
Louisiana	8. 8	65. 8	. 1	. 3	34. 1	28. 7	43. 0	94. 8	40. 5	4. 3	83. 5	99. 1	16. 5	. 9	100. 0	100. 0
Maryland	15. 3	58. 3	20. 2	33. 8	20. 3	3. 9	55. 8	96. 0	29. 3	3. 0	85. 1	99. 0	14. 9	1. 0	100. 0	100. 0
Michigan	5. 9	58. 2	4. 7	9. 7	53. 1	28. 1	63. 7	96. 0	23. 8	3. 3	87. 5	99. 3	12. 5	. 7	100. 0	100. 0
Missouri	3. 4	57. 8	2. 7	12. 3	25. 2	19. 7	31. 3	89. 8	51. 8	9. 3	83. 1	99. 1	16. 9	. 9	100. 0	100. 0
Montana	2. 2	30. 1	4. 5	35. 2	9. 5	12. 4	16. 2	77. 7	5. 2	3. 6	21. 4	81. 3	78. 6	18. 7	100. 0	100. 0
Nevada	. 2	10. 2	10. 0	70. 2	3. 3	3. 8	13. 5	84. 2	3. 7	2. 5	17. 2	86. 7	82. 8	13. 3	100. 0	100. 0
North Dakota	(1)	2. 2	. 6	17. 9	15. 2	59. 6	15. 8	79. 7	17. 5	11. 0	33. 3	90. 7	66. 7	9. 3	100. 0	100. 0
Ohio	9. 8	54. 8	15. 3	24. 9	60. 3	18. 9	85. 4	98. 6	8. 5	. 8	93. 9	99. 4	6. 1	. 6	100. 0	100. 0
Oklahoma	2. 7	45. 9	1. 3	11. 0	11. 0	21. 5	15. 0	78. 4	64. 7	19. 9	79. 7	98. 3	20. 3	1. 7	100. 0	100. 0
Oregon	3. 9	47. 0	6. 2	27. 9	31. 2	20. 9	41. 3	95. 8	32. 3	3. 2	73. 6	99. 0	26. 4	1. 0	100. 0	100. 0
South Dakota	. 2	9. 0	. 9	16. 9	18. 4	53. 6	19. 5	79. 5	40. 5	18. 3	60. 0	97. 8	40. 0	2. 2	100. 0	100. 0
Texas	3. 5	34. 7	6. 7	35. 2	13. 6	10. 5	23. 8	80. 4	8. 6	5. 5	32. 4	85. 9	67. 6	14. 1	100. 0	100. 0
Utah	1. 7	33. 4	6. 9	39. 9	24. 9	19. 7	33. 5	93. 0	17. 0	3. 9	50. 5	96. 9	49. 5	3. 1	100. 0	100. 0
Vermont	6. 6	55. 0	3. 3	15. 7	42. 0	25. 7	51. 9	96. 4	32. 6	3. 2	84. 5	99. 6	15. 5	. 4	100. 0	100. 0
West Virginia	7. 5	50. 4	6. 3	20. 9	18. 8	13. 5	32. 6	84. 8	19. 6	6. 8	52. 2	91. 6	47. 8	8. 4	100. 0	100. 0
Wyomiug	. 1	. 8	11. 4	72. 7	4. 8	7. 6	16. 3	81. 1	4. 8	2. 3	21. 1	83. 4	78. 9	16. 6	100. 0	100. 0
Average	4. 0	47. 3	6. 1	24. 1	24. 8	18. 4	34. 9	89. 8	22. 4	4. 9	57. 3	94. 7	42. 7	5. 3	100. 0	100. 0

[1] Less than 0.05 percent.

carried only 5.3 percent of the total vehicle-mileage. This shows that the improvement of roads now unimproved would mean, in general, the construction of relatively large mileages to serve a relatively small portion of the vehicle-miles of travel. These relationships are presented graphically in figure 2.

Not only do the roads of higher type surface serve greater traffic densities than those of lower type surface, but they also, in general, pass through more densely populated rural areas. Table 5 shows the number per mile of farm units and other rural dwellings along roads with different type surfaces in each of 32 States. The States are grouped into four regions from east to west. The greatest density of dwellings along all rural roads is in the eastern region; the next greatest is in the western region; and the density is lower for the two intermediate regions. For all 32 States, there are, on the average, 2.7 rural dwellings per mile of rural road. The dwelling density along paved roads is 7.1 per mile and is lower for each successively lower general surface type, ranging down to 1.5 per mile for unimproved roads. In individual States, however, the roads of higher surface type do not always have the greater dwelling densities. In Vermont, Michigan, Illinois, Maryland, Missouri, Texas, and Louisiana for example, there are fewer houses per mile along paved roads than along roads having other dustless surfaces, and in North Dakota, Nebraska, Montana, Nevada, and Arizona there are fewer houses per mile along roads having dustless surfaces other than pavement, than along roads having nondustless surfaces. These minor variations in the general trend are caused by the building of high-type roads, important to through traffic, across relatively sparsely settled areas.

Table 5.—*Number of rural dwellings per mile along roads with different general surface types by States (grouped by regions)*

Region and State	Pavement	Other dustless surface	Nondustless surface	Graded and drained	Unimproved	All types
Region 1:						
New Hampshire	10.0	7.8	3.6	2.6	1.2	4.3
Vermont	7.4	8.2	4.0	2.5	1.4	3.5
Michigan	6.3	7.2	4.3	2.8	1.1	3.8
Illinois	4.6	9.8	3.2	2.4	1.7	3.1
Ohio	10.0	7.2	4.2	2.3	2.1	5.0
Maryland	10.0	10.4	6.5	4.3	3.9	6.7
West Virginia	13.9	11.2	6.4	4.0	3.3	5.4
Kentucky	9.2	8.3	5.1	4.5	4.0	5.0
North Carolina	10.8	8.1	5.4	5.5	3.4	5.8
South Carolina	9.1	7.2	5.7	5.9	4.9	5.5
Florida	7.2	4.6	3.1	2.6	2.0	3.4
Average	8.1	7.4	4.3	3.4	3.2	4.5
Region 2:						
North Dakota	1.6	1.0	1.1	.9	.6	.7
South Dakota	1.9	1.6	1.3	.9	.4	.8
Wisconsin	5.3	4.2	3.2	1.9	.9	3.0
Iowa	2.7	2.6	2.6	2.1	1.9	2.3
Nebraska	2.6	1.2	2.0	1.5	1.1	1.3
Missouri	4.0	5.6	3.2	2.5	1.6	2.7
Kansas	6.4	2.4	2.3	1.1	1.2	1.5
Arkansas	5.8	4.7	4.0	3.2	3.1	3.4
Oklahoma	3.5	3.1	2.8	2.2	1.5	2.2
Texas	4.0	4.5	3.8	2.1	2.4	2.7
Louisiana	7.5	9.7	5.9	3.2	2.9	4.4
Average	4.5	4.0	2.9	2.0	1.5	2.1
Region 3:						
Montana	1.9	1.6	1.8	1.0	.7	.9
Idaho	8.2	2.9	2.8	1.4	1.0	1.6
Wyoming	4.1	1.4	1.3	.6	.8	.9
Colorado	4.2	2.3	2.2	1.2	.9	1.2
Utah	9.8	3.6	2.0	1.1	.5	1.3
Nevada	4.8	.9	1.6	.8	.4	.5
Arizona	13.6	3.2	5.4	2.4	1.1	2.2
Average	5.6	2.1	2.3	1.4	.8	1.2
Region 4:						
Washington	9.7	4.1	3.8	1.3	.8	2.8
Oregon	6.6	3.9	3.4	.7	.4	1.9
California	10.8	6.8	2.5	2.0	1.0	3.7
Average	10.1	6.1	3.4	1.1	.9	3.0
Average for all States represented	7.1	5.6	3.5	2.3	1.5	2.7

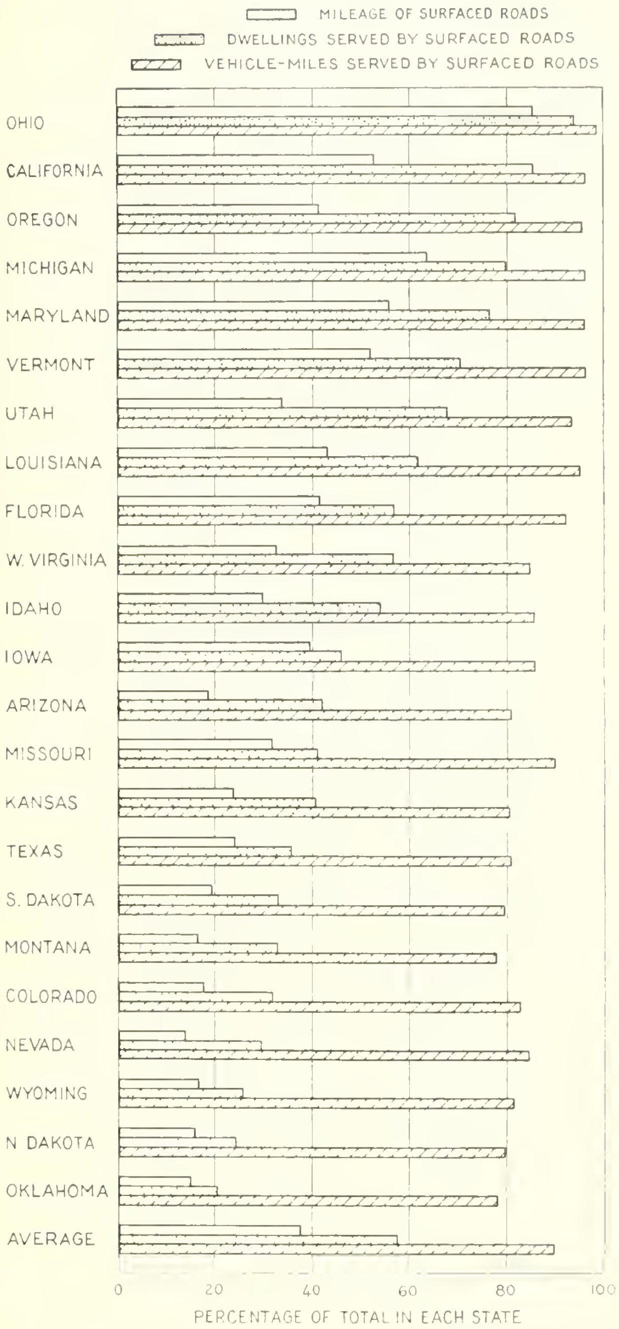

Figure 3.—Percentage of Rural Road Mileage Surfaced, Percentage of All Rural Dwellings Directly Served by Surfaced Roads, and Percentage of Total Travel on Rural Roads, in Each of 23 States.

In table 6, and in figure 3, a comparison is made of the percentage of the rural road mileage surfaced, the percentage of the vehicle-miles of travel served by surfaced roads, and the percentage of the rural dwellings directly served by surfaced roads, in each of 23 States. For these States, 37.8 percent of the total rural road mileage is surfaced, and this surfaced mileage serves directly 57.2 percent of the rural dwellings and accommodates 89.8 percent of the vehicle-miles of travel on rural roads. In Ohio, 93.8 percent of the rural dwellings are directly served by surfaced roads, whereas in

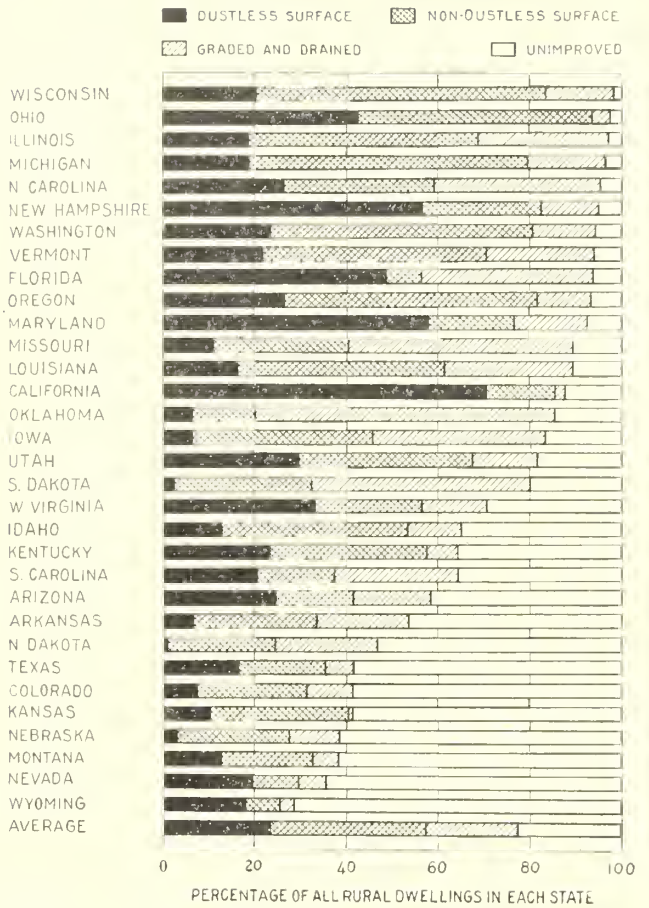

FIGURE 4.—PERCENTAGE OF ALL RURAL DWELLINGS DIRECTLY SERVED BY ROADS WITH DIFFERENT GENERAL SURFACE TYPES IN EACH OF 32 STATES.

Oklahoma only 20.1 percent are so served. The percentages of rural dwellings served by surfaced roads in other States range between these two extremes.

The percentages of all rural dwellings directly served by roads with different general surface types are shown for each of 32 States in table 7 and in figure 4. By omitting vehicle-mileage data, it was possible to include 9 more States in this table than in table 6. In these 32 States, 57.2 percent of all rural dwellings were directly served by surfaced roads, which is exactly the same percentage as that shown in table 6 for 23 States. Only 22.6 percent of the rural dwellings were located on unimproved roads. For individual States, the percentage of rural dwellings located on unimproved roads varied from 71.2 percent in Wyoming down to 1.5 percent in Wisconsin.

TWO-THIRDS OF RURAL DWELLINGS IN 10 STATES LOCATED WITHIN 1 MILE OF SURFACED ROAD

Many rural dwellings which do not front directly on improved roads are located close to them so that the occupants need travel only a short distance to get to an improved highway. Under such conditions, the actual mileage which need be driven on unimproved roads is very small in relation to the total mileage driven on an average trip. In 10 States, studies to determine the number of rural dwellings located within different travel distances of improved roads have been completed.

Table 8 and figure 5 show the percentages of all rural dwellings within various travel distances of surfaced roads, in the 10 States in which this information is available. In these States 65.0 percent of the rural dwellings were within 1 mile of a surfaced road, and 77.5 percent were within 2 miles of a surfaced road.

TABLE 6.—*Percentage of rural road mileage surfaced, and percentage of all rural travel and of all rural dwellings directly served by surfaced roads in each of 23 States*

State	Percentage of total rural road mileage surfaced	Percentage of total rural vehicle-mileage served by surfaced roads	Percentage of all rural dwellings directly served by surfaced roads
Arizona	18.3	80.4	41.7
California	52.9	96.2	85.7
Colorado	17.3	82.3	31.5
Florida	41.1	92.0	56.7
Idaho	29.5	85.8	53.5
Iowa	39.1	85.7	45.8
Kansas	23.3	80.5	40.2
Louisiana	43.0	94.8	61.2
Maryland	55.8	96.0	76.2
Michigan	63.7	96.0	79.8
Missouri	31.3	89.8	40.9
Montana	16.2	77.7	32.4
Nevada	13.5	84.2	29.8
North Dakota	15.8	79.7	24.2
Ohio	85.4	98.6	93.8
Oklahoma	15.0	78.4	20.1
Oregon	41.3	95.8	81.5
South Dakota	19.5	79.5	32.8
Texas	23.8	80.4	35.2
Utah	33.5	93.0	67.4
Vermont	51.9	96.4	70.3
West Virginia	32.6	84.8	56.1
Wyoming	16.3	81.1	25.4
Average	37.8	89.8	57.2

TABLE 7.—*Percentage of all rural dwellings directly served by roads with different general surface types, in each of 32 States*

State	Pavement	Other dustless surfaces	Non-dustless surfaces	All surfaced roads	Graded and drained roads	All improved roads	Unimproved roads
Arizona	12.6	11.7	17.4	41.7	16.4	58.1	41.9
Arkansas	5.0	1.6	27.1	33.7	19.7	53.4	46.6
California	29.8	40.3	15.6	85.7	2.1	87.8	12.2
Colorado	2.2	5.3	24.0	31.5	9.9	41.4	58.6
Florida	13.4	35.3	8.0	56.7	36.9	93.6	6.4
Idaho	2.6	10.2	40.7	53.5	11.5	65.0	35.0
Illinois	16.7	2.1	49.7	68.5	28.7	97.2	2.8
Iowa	5.7	.6	39.5	45.8	37.4	83.2	16.8
Kansas	6.0	4.2	30.0	40.2	1.1	41.3	58.7
Kentucky	6.3	17.0	34.6	57.9	6.4	64.3	35.7
Louisiana	16.3	.1	44.8	61.2	28.3	89.5	10.5
Maryland	24.3	33.6	18.3	76.2	16.4	92.6	7.4
Michigan	9.9	8.9	61.0	79.8	16.7	96.5	3.5
Missouri	5.2	5.6	30.1	40.9	48.8	89.7	10.3
Montana	4.8	8.1	19.5	32.4	5.8	38.2	61.8
Nebraska	2.0	.8	25.1	27.9	10.7	38.6	61.4
Nevada	2.1	17.3	10.4	29.8	5.6	35.4	64.6
New Hampshire	7.6	48.9	25.9	82.4	12.9	95.3	4.7
North Carolina	15.7	10.4	32.9	59.0	36.9	95.9	4.1
North Dakota	.1	.8	23.3	24.2	22.7	46.9	53.1
Ohio	20.0	22.3	51.5	93.8	3.8	97.6	2.4
Oklahoma	4.3	1.9	13.9	20.1	65.5	85.6	14.4
Oregon	13.4	12.7	55.4	81.5	11.9	93.4	6.6
South Carolina	8.9	11.7	16.6	37.2	27.1	64.3	35.7
South Dakota	.6	1.8	30.4	32.8	47.2	80.0	20.0
Texas	5.2	11.0	19.0	35.2	6.5	41.7	58.3
Utah	10.0	19.9	37.5	67.4	14.2	81.6	18.4
Vermont	14.2	7.7	48.4	70.3	23.6	93.9	6.1
Washington	14.2	9.3	57.4	80.9	13.2	94.1	5.9
West Virginia	19.6	13.4	23.1	56.1	14.8	70.9	29.1
Wisconsin	10.7	10.0	62.6	83.3	15.2	98.5	1.5
Wyoming	.3	17.7	7.4	25.4	3.4	28.8	71.2
Average	11.3	12.2	33.7	57.2	20.2	77.4	22.6

(Continued on page 158)

GRAPHICAL ANALYSES OF THE STABILITY OF SOIL

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by E. S. BARBER, Junior Highway Engineer, and C. E. MERSHON, Junior Engineer

TO FURTHER the development of rational procedures for use in the design and construction of highways during the past two decades, the Public Roads Administration has made comprehensive studies of published material and conducted supplementary laboratory research on the stability of soil. Results of this work, published in PUBLIC ROADS and in the Proceedings of the Highway Research Board, include the interpretation of test data, the evaluation of pressure against retaining walls, the design of cuts and embankment cross sections, and the estimation of the supporting value of undersoil.

In an effort to expedite a general use of the theories as a basis of correlation with experience, this report presents methods of analyses in which charts are used to facilitate computations and thus greatly reduce the time and labor required in the application of the formulas. A summary of the development of the formulas and a brief discussion of the assumptions on which the theories are based are first presented. An explanation of the construction and use of the charts then follows.

STABILITY OF SOIL DEPENDS UPON ITS SHEARING RESISTANCE

The stability of a soil is assumed to depend upon its shearing resistance which, according to Coulomb's classical theory published in 1773 (*1*),[1] is expressed by the relation

$$s=c+n \tan \phi \quad \text{(1)}$$

in which

s = unit shearing resistance,

c = unit cohesion,

n = stress normal to the plane of shear, and

ϕ = angle of internal friction.

Cohesion is defined as that component of shearing resistance which is independent of the stress normal to the plane of shear. (See fig. 1.) Internal friction is defined as that component of shearing resistance which is directly proportional to the stress normal to the plane of shear.

A factor of safety with respect to total strength may be applied by dividing c and $\tan \phi$ by the desired factor (*2*). For a deformation less than that at failure, the corresponding c and ϕ (*3*) may be used by assuming a hypothetical soil with these ultimate values.

Compressive strength.—The relation between the unit compressive strength, v_0, of an unconfined cylindrical soil sample, its cohesion, and its angle of internal friction, was discussed in the Proceedings of the Nineteenth Annual Meeting of the Highway Research Board (*4*). The formula is

$$v_0=2c \tan \alpha \quad \text{(2)}$$

where

$$\alpha=45°+\frac{\phi}{2}$$

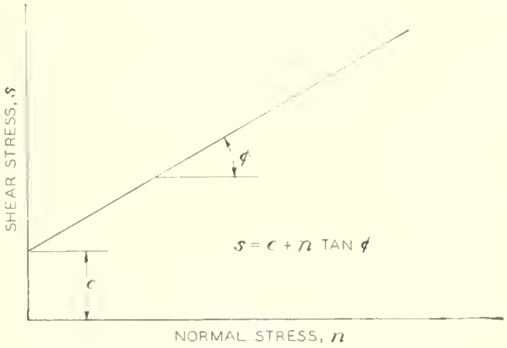

FIGURE 1.—RELATION OF SHEAR STRESS TO NORMAL STRESS IN A SOIL SAMPLE AT FAILURE.

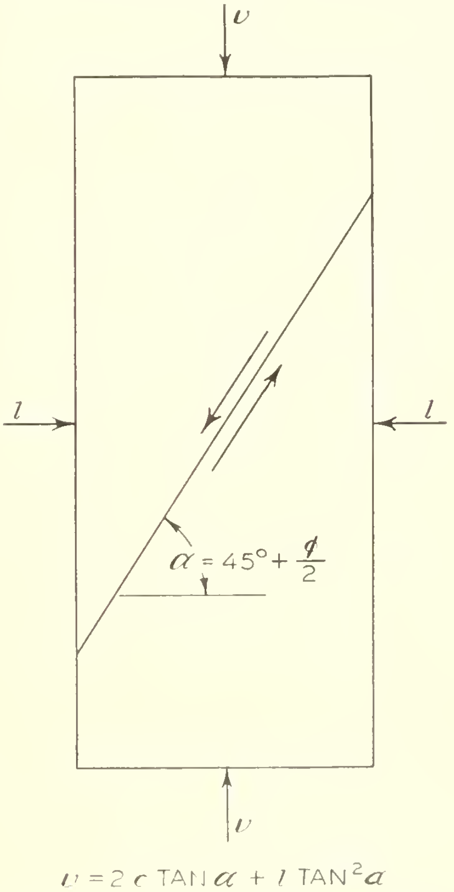

FIGURE 2.—RELATION OF PRINCIPAL STRESSES AT FAILURE OF ANY POINT IN A STRESSED EARTH MASS.

If a unit lateral pressure, l, is applied to a sample as in figure 2, the expression for its unit compressive strength, v, as published in PUBLIC ROADS, December 1938 (*5*), becomes

$$v=2c \tan\alpha+l \tan^2\alpha \quad \text{(3)}$$

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[1] Italic figures in parentheses refer to bibliography, page 155.

Tangent functions of α and its complement β are given in table 1.

The relation between the maximum height, H, at which an unrestrained embankment will stand vertically, the unit weight of the soil, w, and its shearing resistance, was discussed in PUBLIC ROADS, December 1929 (*6*). The expression is

$$H=\frac{c_0}{w}=\frac{2c}{w}\tan\alpha \quad (4)$$

TABLE 1.—*Tangent functions of α and β*

ϕ, degrees	$\tan\phi$	$\alpha=45^\circ+\frac{\phi}{2}$		$\beta=45^\circ-\frac{\phi}{2}$	
		$\tan\alpha=\cot\beta$	$\tan^2\alpha=\cot^2\beta$	$\tan\beta=\cot\alpha$	$\tan^2\beta=\cot^2\alpha$
0	0	1.000	1.000	1.000	1.000
1	0.017	1.018	1.036	.983	.966
2	.035	1.036	1.072	.966	.933
3	.052	1.054	1.110	.949	.901
4	.070	1.072	1.150	.933	.870
5	.087	1.091	1.191	.916	.840
6	.105	1.111	1.233	.900	.811
7	.123	1.130	1.278	.885	.783
8	.141	1.150	1.323	.869	.756
9	.158	1.171	1.371	.854	.729
10	.176	1.192	1.420	.839	.704
11	.194	1.213	1.472	.824	.680
12	.213	1.235	1.525	.810	.656
13	.231	1.257	1.580	.795	.633
14	.249	1.280	1.638	.781	.610
15	.268	1.303	1.698	.767	.589
16	.287	1.327	1.761	.754	.568
17	.306	1.351	1.826	.740	.548
18	.325	1.376	1.894	.727	.528
19	.344	1.402	1.965	.713	.509
20	.364	1.428	2.040	.700	.490
21	.384	1.455	2.117	.687	.472
22	.404	1.483	2.198	.675	.455
23	.424	1.511	2.283	.662	.438
24	.445	1.540	2.371	.649	.422
25	.466	1.570	2.464	.637	.406
26	.488	1.600	2.561	.625	.390
27	.510	1.632	2.663	.613	.376
28	.532	1.664	2.770	.601	.361
29	.554	1.698	2.882	.589	.347
30	.577	1.732	3.000	.577	.333
31	.601	1.767	3.124	.566	.320
32	.625	1.804	3.255	.554	.307
33	.649	1.842	3.392	.543	.295
34	.675	1.881	3.537	.532	.283
35	.700	1.921	3.690	.521	.271
36	.727	1.963	3.852	.510	.260
37	.754	2.006	4.023	.499	.219
38	.781	2.050	4.294	.488	.238
39	.810	2.097	4.395	.477	.228
40	.839	2.145	4.599	.466	.217
41	.869	2.194	4.815	.456	.208
42	.900	2.246	5.045	.445	.198
43	.933	2.300	5.289	.435	.189
44	.966	2.356	5.550	.424	.180
45	1.000	2.414	5.828	.414	.172

Active and passive pressures.—Formulas for finding the lateral pressures of soils against retaining walls were published in PUBLIC ROADS, December 1938 (*5*). For the simplest case of a cohesive soil with level backfill, vertical back of wall, no surcharge, and swelling phenomena neglected, the total active horizontal pressure, L, per unit length of wall is obtained from the expression

$$L=h\left(\frac{wh}{2}\tan^2\beta-2c\tan\beta\right) \quad (5)$$

in which

h=height of backfill, and

$\beta=45^\circ-\frac{\phi}{2}$.

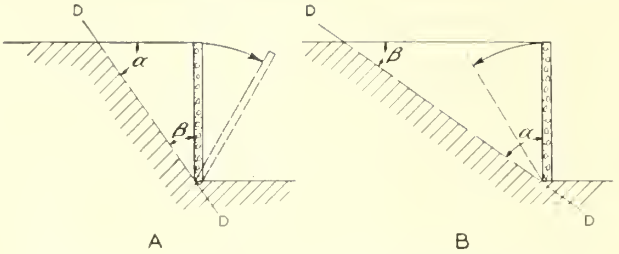


FIGURE 3.—SURFACES OF SLIP BEHIND WALLS.

For the same conditions, the total passive earth pressure, P, per unit length of wall is given by the formula

$$P=h\left(\frac{wh}{2}\tan^2\alpha+2c\tan\alpha\right) \quad (6)$$

The significance of the terms "active" and "passive" earth pressure has been described in PUBLIC ROADS (*3*) as follows:

> In the design of retaining walls, three types of earth pressure may be considered.
>
> Without movement of the earth, pressures against the walls, figures 3-A and 3-B, become the "earth pressures at rest" which depend upon the coefficient K, expressed by the relation: $K=l/v$. (K depends on the soil's elasticity.)
>
> However, soil must deform to fail. The pressures it produces at maximum deformation without failure are termed active or passive, depending on the directions of the applied forces responsible.
>
> Wedges assumed in the design of retaining walls, figure 3, have lower boundaries, D–D, on which the soil slips when it shears. Weight of the earth in figure 3–A produces the active earth pressure which forces walls outward and causes D–D to incline at an angle α with the horizontal and β with the vertical. Forcing walls backward as in figure 3–B produces the passive earth pressure which causes D–D to incline at an angle β with the horizontal and α with the vertical.

A cable anchorage would exert passive pressure on the soil in front of it. The surface shear test apparatus developed by Burggraf (*7*) measures a similar passive resistance.

FORMULAS GIVEN FOR BEARING CAPACITY AND DISTORTION OF SOILS

Bearing capacity under strip load.—It will be noted that equations 5 and 6 have two parts. The first depends on the weight of the earth in the wedge and the second on the cohesion. As shown in figure 4–A, the bearing capacity, q, of soil under a long, uniform, strip load depends on an active wedge being held in equilibrium by a passive wedge. Since the passive pressure, P, is always greater than the active pressure, L, an additional pressure, q, can be supported at the surface of the active wedge. Then at equilibrium

$$q=\left(\frac{P-L}{h}\right)\tan^2\alpha \quad (7)$$

By substituting equations 5 and 6 for L and P in equation 7 there is obtained

$$q=w_fH=c\frac{2\tan\alpha}{\cos^2\alpha}+w_uB\frac{\tan^4\alpha-1}{2\cot\alpha} \quad (8)$$

in which

w_f=unit weight of embankment material,
H=critical height of embankment,
w_u=unit weight of undersoil, and
B=width of active wedge at the subgrade surface, figure 4–A.

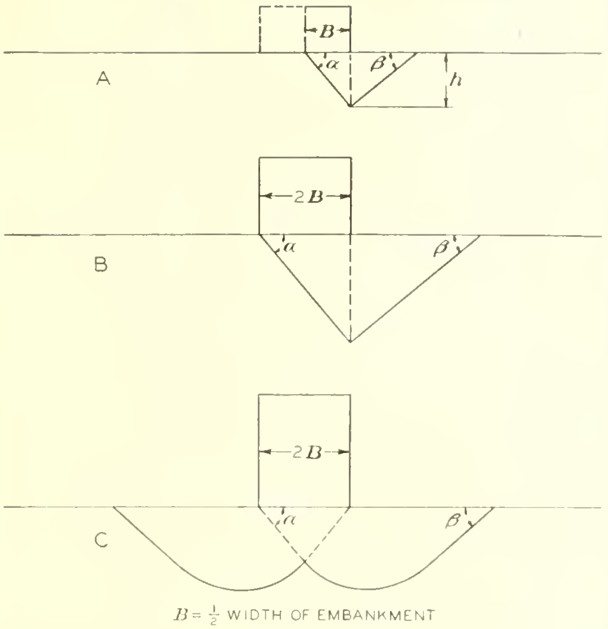


Figure 4.—Surfaces of Slip Under Embankments ($\phi = 10°$).

If a surcharge of thickness T is applied to the surface of the passive wedge, figure 5–A, the load on the active wedge may be increased by $w_sT \tan^4 \alpha$, where w_s is the unit weight of the surcharge material. The total bearing capacity (*8*) then becomes

$$q = w_fH = c\,\frac{2 \tan \alpha}{\cos^2 \alpha} + w_uB\,\frac{\tan^4\alpha - 1}{2 \cot \alpha} + w_sT \tan^4 \alpha \text{ -- (9)}$$

For zero cohesion, the first term is zero; for zero friction, the second term is zero; and for zero surcharge, the third term is zero. If B is taken as zero in equation 9, the bearing capacity becomes equivalent to

$$q = 2c\,(\tan^3 \alpha + \tan \alpha) + w_sT \tan^4 \alpha \text{ ------ (10)}$$

which gives the maximum allowable vertical pressure under the edge of a footing (*9*). Equation 9 has been suggested for use in estimating the supporting value of a homogeneous subgrade under a symmetrical strip load which divides in the center as it fails.

To apply the formula to a long fill, the cross section of the fill must be modified. With reference to figure 5, it should be noted that B is the width of the active wedge whereas b is one-half the top width of the fill. One method of solution is to assume a rectangular cross section of width $2B$ and an area equal to the area of the fill as shown by figure 5–B. This solution considers no surcharge. Another method is to assume a rectangle of width $2B$ with equal surcharges on each side as shown by figure 5–C. The area of the rectangle plus the area of the surcharges is equal to the actual area of the fill cross section.

If the embankment can be considered rigid enough to settle as a unit and the undersoil moves out on one side only as in figure 4–B, the full width, $2B$, is used in place of B and equation 9 becomes

$$q = c\,\frac{2 \tan \alpha}{\cos^2 \alpha} + w_uB\,\frac{\tan^4 \alpha - 1}{\cot \alpha} + w_sT \tan^4 \alpha \text{ ----- (11)}$$

A different problem is presented by a fill, figure 4–C, which is rigid enough to settle vertically without tilting and without breaking in the middle, forcing the undersoil out on both sides. Prandtl's formula with a term added to include the weight of the supporting soil (*10*) may be used to find the supporting power of the undersoil under these circumstances. The formula considers no surcharge and is

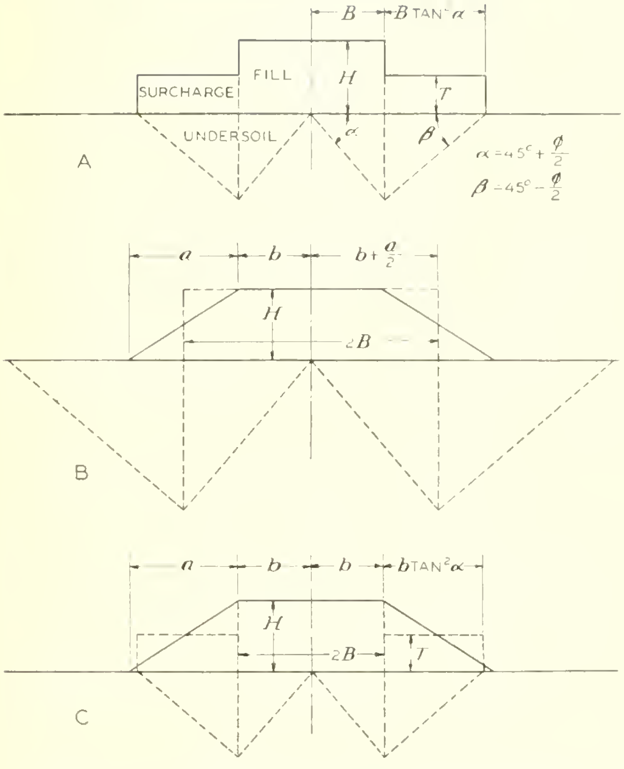


Figure 5.—Modified Embankment Cross Sections.

$$q = (c \cot \phi + w_uB \tan \alpha)\,(\tan^2 \alpha / e^{\pi \tan \phi} - 1) \text{ - (12)}$$

Distortion of soil. If the amount of distortion in the soil behind a retaining wall were relatively the same as in a shear sample and the wedge of soil behind a rotating wall deformed in pure shear parallel to the plane of failure, figure 6, the average movement of the top of the soil wedge h feet high as the wall rotates about its base (*4*) would be:

For walls moving out (active pressure), figure 6–A.

$$d_l = \frac{mh}{200} \sin^2 \beta \text{ feet} = 0.03mh(1 - \sin \phi) \text{ inches --- (13)}$$

$$d_v = \frac{mh}{200} \sin \beta \cos \beta \text{ feet} = 0.03mh \cos \phi \text{ inches --- (14)}$$

For walls moving in (passive pressure), figure 6–B.

$$d_l = \frac{mh}{200} \cos^2 \beta \text{ feet} = 0.03mh(1 + \sin \phi) \text{ inches --- (15)}$$

$$d_v = \frac{mh}{200} \sin \beta \cos \beta \text{ feet} = 0.03mh \cos \phi \text{ inches --- (16)}$$

where

d_l = average lateral soil movement,

d_v = average vertical soil movement, and

m = shear strain in percent $= \dfrac{\text{tangential movement}}{\text{thickness}} \times 100.$

In a direct shear test, the shear strain may be taken as the shear movement divided by the effective thickness of the sample. For constant volume and small strains the maximum shear strain in a uniformly stressed cylinder is 1.5 times the vertical strain (*11*). The average

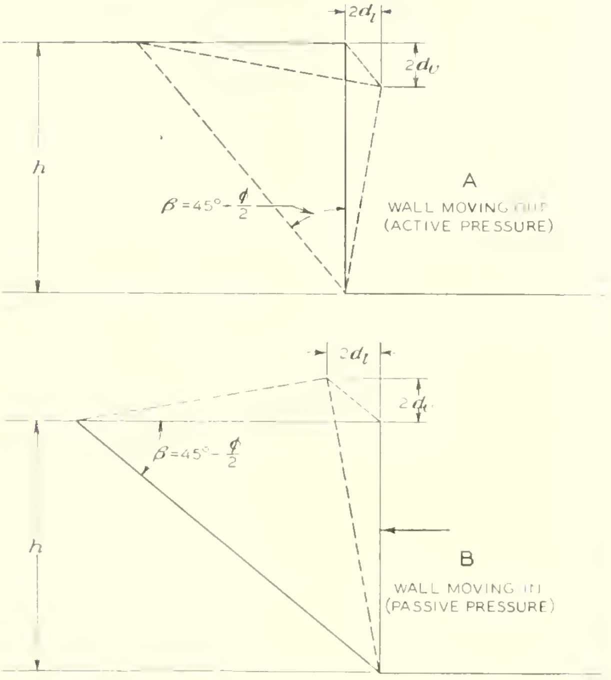


FIGURE 6.—DISTORTION OF SOIL BEHIND WALL.

settlement due to lateral movement of the soil supporting a long nonrigid embankment may be estimated by using B, the half width of the fill, in place of h in equation 15.

Bearing capacity of thin layer.—For a plastic material squeezed between two parallel rigid plates, the theory of plasticity indicates that the bearing capacity varies directly as the shearing resistance and as the ratio of the width of the plates to the distance between them if this ratio is at least four (*12, 13*). For the case of a triangular load of relatively firm material resting on a soft layer which is underlaid by relatively firm material, figure 7-A, the expression for the unit bearing capacity, q, of the soft undersoil is

$$q-W_fH_0=s\frac{x}{D}\qquad(17)$$

where

w_f= unit weight of fill material,
H_0= height of triangular fill,
s= unit shearing resistance of soft layer,
x= width of fill at base, and
D= depth of soft layer.

By assuming a triangular embankment, figure 7-A, and substituting $c+\frac{w_uD}{2}\tan\phi$ (that is, $c+n\tan\phi$ of the soft undersoil) for s and $2H_0S_0$ for x, a formula was derived for calculating the critical slope, S_0, of the embankment. The expression thus obtained is

$$S_0=\frac{Dw_f}{2c+Dw_u\tan\phi}\qquad(18)$$

in which

w_u= unit weight of soft undersoil.

The critical slope of a trapezoidal fill may be estimated from equation 18 by considering a triangle, figure 7-A, with the same base and slope as the embankment. In the case of low, wide fills, the results obtained by this procedure may be too conservative. Another method (*13*) is to consider a triangle with an area equal to the trapezoidal area of the fill as in figure 7-B. For this assumption, the relation between the slope, S, of the fill and the slope, S_0, of the triangle is

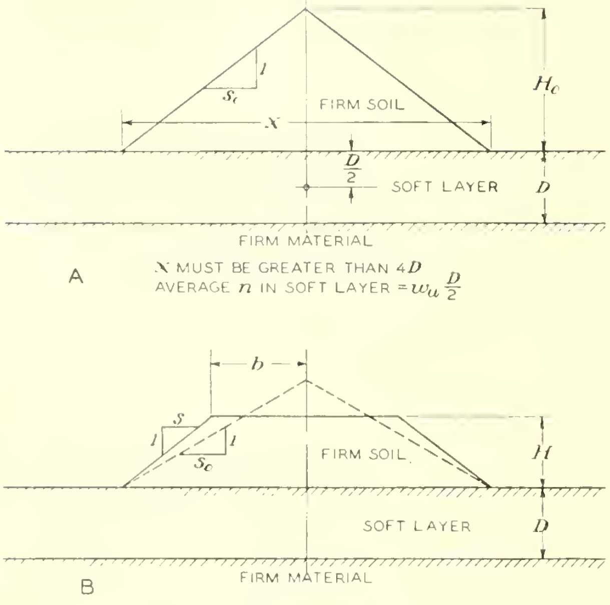


FIGURE 7.—FILL ON SOFT LAYER.

$$S_0=S+\frac{\left(\frac{b}{H}\right)^2}{2\frac{b}{H}+S}\qquad(19)$$

where

b= top half-width of fill, and
H= height of fill.

FORMULA FOR GREATEST SHEARING STRESS UNDER FILL PRESENTED

Critical height of slopes.—A graphical method for determining the critical height of slopes was published in PUBLIC ROADS, December 1929 (*6*). It assumes a circular surface of sliding as shown in figure 8 and compares the moment of the shear resistance along this surface with the moment of the weight of soil bounded by the surface. Moments are taken about the center of curvature. The most dangerous circle is determined by trial. Various analyses of the critical height, H, of cuts and embankments of homogenous material with level tops have been compared and tabulated by Taylor (*14*). His tables give the dimensionless ratio $\frac{c}{wH}$ for various values of ϕ and slope angle i. For a vertical slope, the critical height becomes approximately $\frac{3.83c}{w}$ $\tan\alpha$ which is greater than that given by equation 4, based on different assumptions.

Greatest shear stress under fill.—Applying the theory of elasticity to a semi-infinite, homogeneous, isotropic material, it is found that the greatest shear stress, s_G, under a symmetrical, trapezoidal strip load, figure 9, is on the centerline and is expressed by the formula (*15, 5*)

$$s_G=\frac{2.3zp}{\pi a}\log\frac{z^2+(a+b)^2}{z^2+b^2}\qquad(20)$$

where

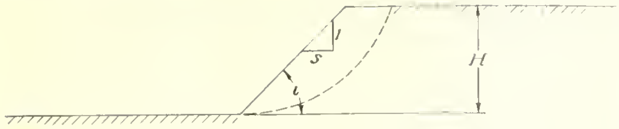


FIGURE 8.—SLIDING SURFACE IN HOMOGENEOUS SLOPE.

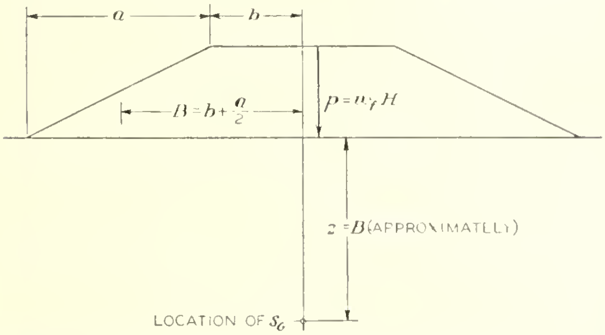


FIGURE 9.—TRAPEZOIDAL LOAD ON ELASTIC MATERIAL.

z = depth below the surface, chosen to make s_G a maximum,
p = pressure on centerline at the surface of the supporting material,
a = width of one side slope of trapezoidal fill,
b = half width of fill at top, and

$$B = b + \frac{a}{2} = z \text{ (approximately).}$$

For the special case of a rectangular strip load, $a = 0$, the shear stress is maximum and equal to p/π at all points on the circumference of a semicircle passing through the edge of the load. If there is a rigid layer at some depth below the load, then according to D. L. Holl (*16*), the greatest shear stress is nearer the surface and of greater magnitude than for a homogeneous supporting material of infinite depth.

The load producing a greatest shearing stress equal to the cohesion of the supporting soil is less than the ultimate bearing capacity, computed for conditions of failure. That is, the load which causes failure at a single point or localized region is less than the load which will cause total failure throughout the supporting soil. Design based on stresses causing failure at restricted regions under a finite area is illustrated by considering the stresses under a circular load.

Stresses under loaded circular areas.—A complete analysis of the stresses below a uniformly loaded circular area, using the theory of elasticity, has been presented by Love (*17, 18*) and includes a tabulation of stresses for Poisson's ratio equal to one-fourth. This analysis was discussed in PUBLIC ROADS (*19*). Figure 10 shows the location and magnitude of the greatest shear stress and the corresponding major principal stress at any level in the undersoil. The influence of Poisson's ratio on these stresses is also shown. The greatest shear stress anywhere under a uniformly loaded circular area is at the surface, just beneath the perimeter. For a Poisson's ratio of one-half, the greatest shearing stress is on the axis for all depths where z/r is greater than 0.7. Broken lines in figure 10 were interpolated. Figure 11 shows the effect upon the greatest shearing stress at any level of varying the applied load when the total load or the area of the circle or the unit pressure is kept

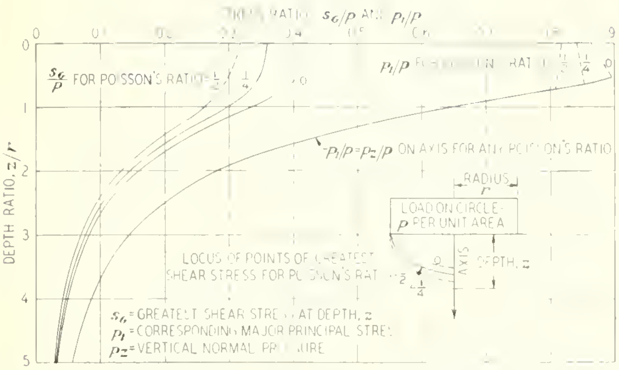


FIGURE 10.—GREATEST STRESSES DUE TO UNIFORM CIRCULAR LOAD ON A SEMI-INFINITE SOLID FOR DIFFERENT POISSON'S RATIOS.

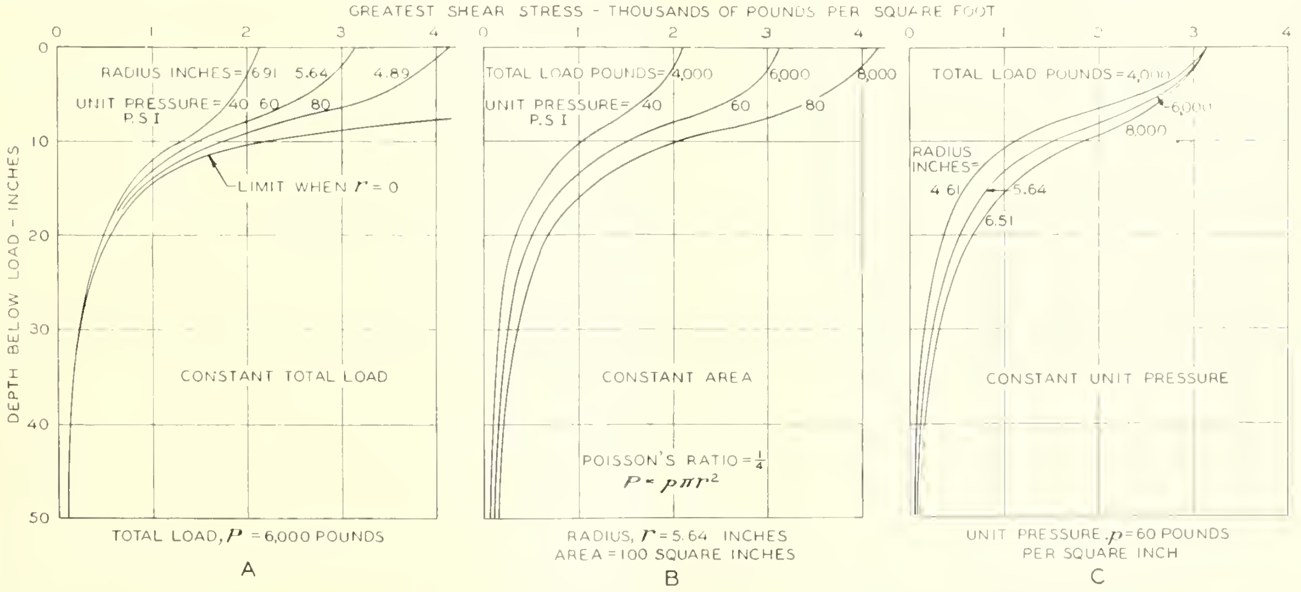


FIGURE 11.—GREATEST SHEAR STRESSES DUE TO UNIFORM CIRCULAR LOAD ON A SEMI-INFINITE SOLID.

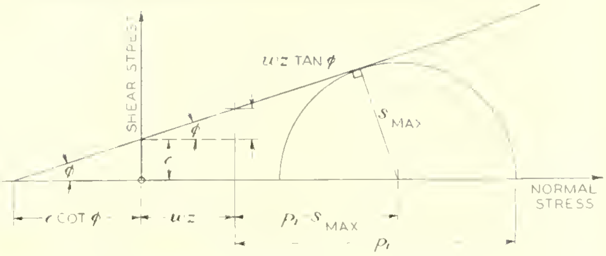


Figure 12.—Relation of Strength to Stresses at a Point.

constant. For ϕ equal to zero and Poisson's ratio equal to one-half, a point at the edge of the load is overstressed if the unit load exceeds 3.14c (see fig. 10) whereas, according to H. Hencky (*12*), the ultimate bearing capacity under a rigid circular load is not reached until the average unit pressure is 5.64c.

To determine the cohesion required to prevent overstress at any point at a given level below a uniformly loaded circular area in a material wherein ϕ is greater than zero, account must be taken of the weight, w, of the material and the normal stresses produced by the load (*20*). Mohr's diagram, figure 12, shows the cohesion required to prevent overstressing at any point and was used in deriving the formula,

$$c = pF - wz \tan \phi \qquad (21)$$

where

$$F = \frac{s_{\max}}{p \cos \phi} - \left(\frac{p_1 - s_{\max}}{p}\right) \tan \phi$$

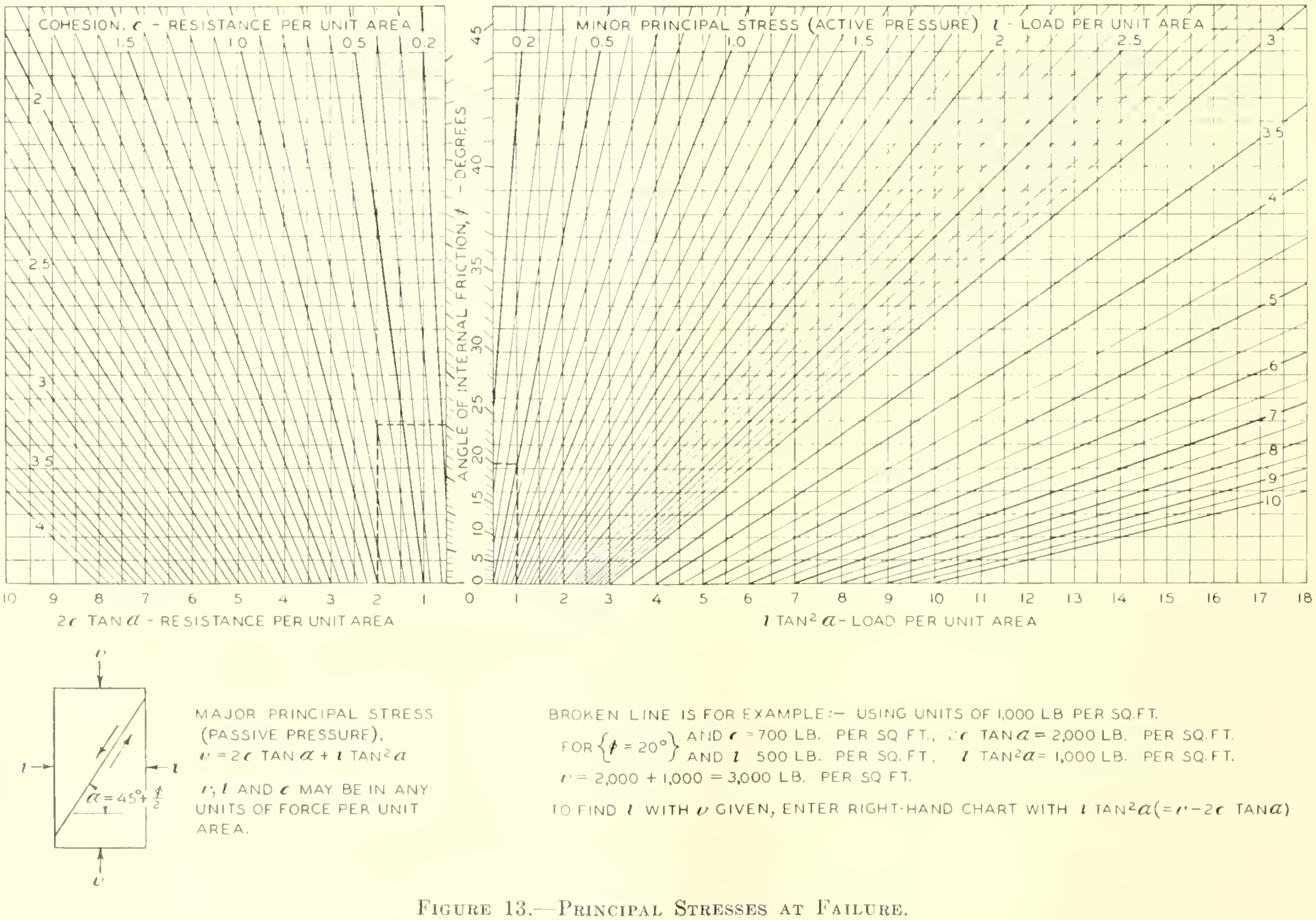
in which $s_{\max}$ and p_1 are the stresses s_G and p_1 shown in figure 10 except for certain values of ϕ for which stresses at points off the axis required the greatest cohesion.

This analysis may be useful as a qualitative indication of the shearing strength required in flexible pavements and subgrades under pneumatic tire loads. However, as in other soil stability problems, due consideration must be given to such factors as wetting and drying, freezing and thawing, swelling and consolidation, distortion, and nonuniformity.

CHARTS USED TO FACILITATE COMPUTATIONS

The charts used in the solution of the foregoing formulas are of the simplest types and, in general, permit the determination of any one variable if the others are given (*21*). Supplementing the method of constructing each chart is an illustrative example which demonstrates its use.

Principal stresses at failure.—If equal lateral and vertical pressures are applied to a right circular cylinder and the vertical pressure then increased to failure, these stresses are related to c and ϕ by equation 3. This formula may be used to determine either the major principal stress, v, or the minor principal stress, l. To construct the graph, each term was evaluated separately. Thus, in the left chart of figure 13, tan α is plotted on the vertical axis, marked with the corresponding values of ϕ, and is multiplied by the sloping lines for various values of c to determine 2c tan α on the horizontal axis. On the right chart tan$^2 \alpha$ is plotted on the vertical scale. An illustrative problem in which c, ϕ, and l are given is shown on the figure.



MAJOR PRINCIPAL STRESS
(PASSIVE PRESSURE),
$v = 2c \text{ TAN } \alpha + l \text{ TAN}^2 \alpha$

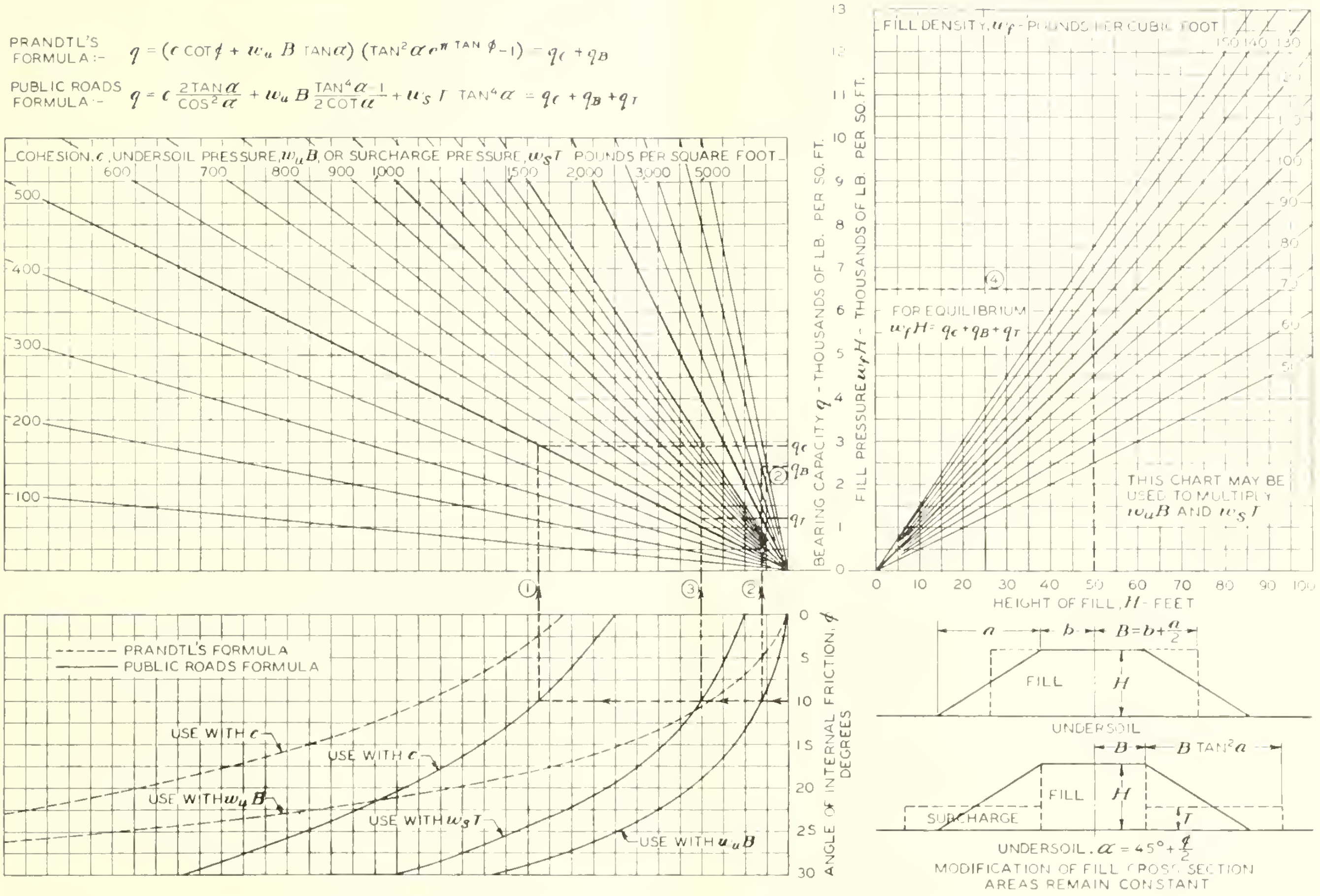
v, l AND c MAY BE IN ANY UNITS OF FORCE PER UNIT AREA.

BROKEN LINE IS FOR EXAMPLE:— USING UNITS OF 1,000 LB PER SQ.FT.

FOR $\{\phi = 20°\}$ AND c = 700 LB. PER SQ FT., $2c$ TAN α = 2,000 LB. PER SQ.FT.
AND l 500 LB. PER SQ. FT., l TAN$^2\alpha$ = 1,000 LB. PER SQ.FT.
v = 2,000 + 1,000 = 3,000 LB. PER SQ FT.

TO FIND l WITH v GIVEN, ENTER RIGHT-HAND CHART WITH l TAN$^2\alpha$ ($= v - 2c$ TAN α)

Figure 13.—Principal Stresses at Failure.



PRANDTL'S FORMULA:- $q = (c \cot \phi + w_u B \tan \alpha)(\tan^2 \alpha\, e^{\pi \tan \phi} - 1) = q_c + q_B$

PUBLIC ROADS FORMULA:- $q = c\,\frac{2 \tan \alpha}{\cos^2 \alpha} + w_u B\,\frac{\tan^4 \alpha - 1}{2 \cot \alpha} + w_s T \tan^4 \alpha = q_c + q_B + q_T$

STRAIGHT? BROKEN LINES ARE FOR EXAMPLE - BY PUBLIC ROADS FORMULA

FOR $\phi = 10°$ (SAME FOR EACH q) AND $c = 500$ LB. PER SQ. FT., ① GIVES $q_c = 2{,}900$ LB. PER SQ FT.

FOR $B = 40$ FT. AND $w_u = 100$ LB. PER CU. FT, $w_uB = 4{,}000$ LB. PER SQ FT, ② GIVES $q_B = 2{,}400$ LB. PER SQ FT.

FOR $T = 5$ FT. AND $w_s = 120$ LB. PER CU. FT, $w_sT = 600$ LB. PER SQ. FT, ③ GIVES $q_T = 1{,}200$ LB. PER SQ FT.

THEN $q = q_c + q_B + q_T = 6{,}500$ LB. PER SQ FT $= w_fH$ (EQUILIBRIUM); FOR $w_f = 130$ LB. PER CU. FT, ④ $H = 50$ FT.

Figure 14. —Bearing Capacity of Soil Under Long Fill.

To solve for v, start at $\phi = 20°$ and follow the dotted lines to obtain the two terms of equation 3 which are added to determine v. The necessary additions or subtractions as well as the reading of the charts may be done with the aid of temporary marks on a straightedge.

Equation 5 may be rewritten in the form

$$\frac{wh}{2} = 2c \tan \alpha + \frac{L}{h} \tan^2 \alpha$$

The active horizontal thrust, L, against a retaining wall may then be determined from figure 13 by using $\frac{wh}{2}$ for v and $\frac{L}{h}$ for l.

As an example, find L for a wall 20 feet high; given $w = 100$ pounds per cubic foot, $c = 200$ pounds per square foot, $\phi = 10°$, $\frac{wh}{2} = \frac{100 \times 20}{2} = 1{,}000$ pounds, per square foot.

From the left chart, $2c \tan \alpha = 480$ pounds per square foot. Then $\frac{L}{h} \tan^2 \alpha = 1{,}000 - 480 = 520$ pounds per square foot.

From the right-hand chart for $\tan^2 \alpha = 520$ and $\phi = 10°$, $\frac{L}{h} = 370$ pounds per square foot.

Thus $L = 7{,}400$ pounds per foot of wall.

Similarly, the passive pressure, P, back of a retaining wall as expressed by equation 6, may be obtained from figure 13 by substituting P/h for v and $wh/2$ for l. A nomograph of the general formula for pressures on a wall with a cohesionless backfill has been published by Taylor (*22*).

Bearing capacity of soil under long fill.—Figure 14 may used to compute the bearing capacity of a homogeneous subgrade under a symmetrical strip load The figure solves equations 9, 11, and 12 by dividing the total bearing capacity, q, into components—q_c involving c; q_B involving the pressure in the supporting soil, w_uB; and q_T involving the surcharge pressure w_sT. The total bearing capacity is the sum of the components. The lower-left chart gives the value of a function of ϕ and the upper-left chart multiplies this value by the appropriate values of c, w_uB, or w_sT. The values of w_u, w_s, and w_f which are selected should represent the most unfavorable conditions to be anticipated.

The illustrative example shown on figure 14 considers a nonrigid fill with surcharges for which c, ϕ, w_u, w_s, T, w_f, and B are given. In the problem H is solved for by means of equation 9.

Values for a multiplying factor not on the chart such as c equals 50 may be determined by using the line for c

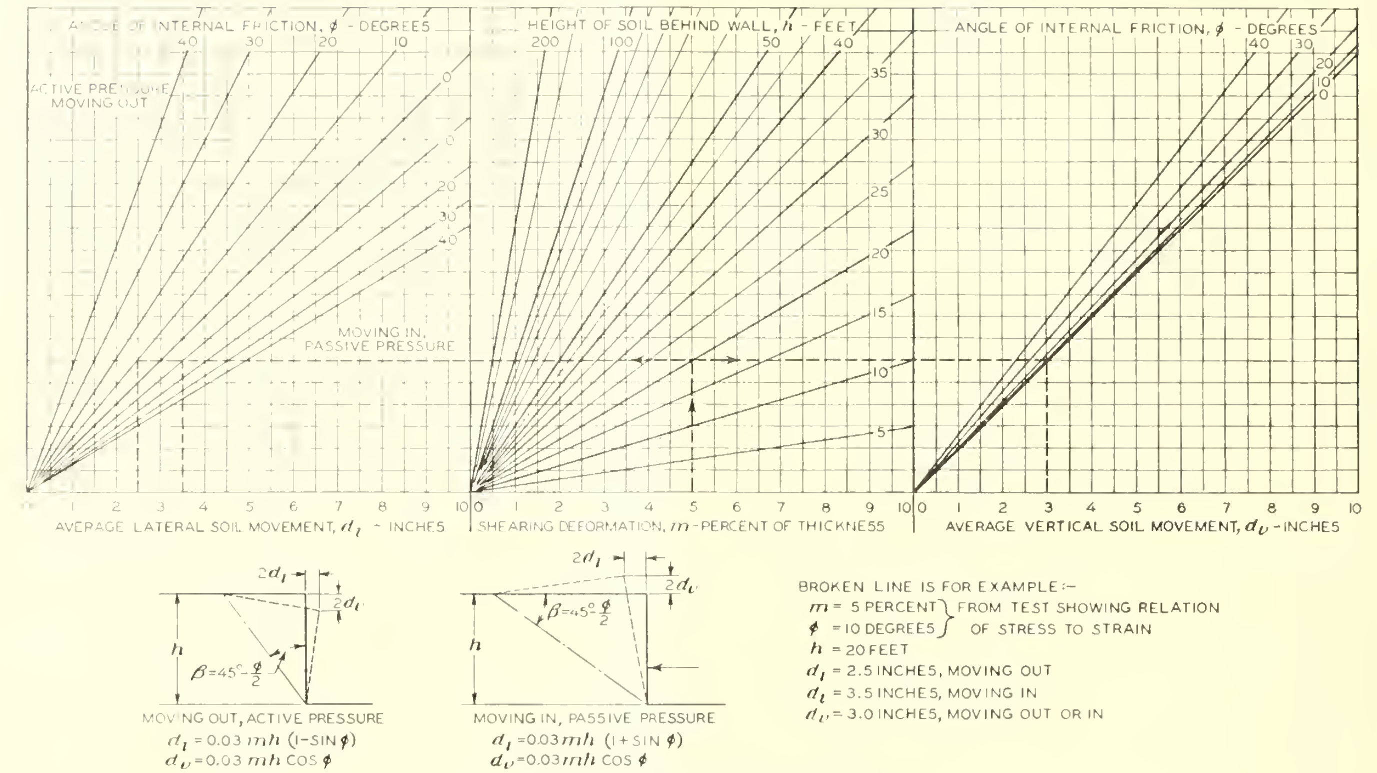


FIGURE 15.—SOIL MOVEMENT BEHIND WALL.

equals x times 50 and dividing the resulting q_c by x. This same device may also be used for w_uB, w_sT, H, w_f, or similar factors on other charts.

Soil movement behind wall.—The movement of the faces of the wedge of soil behind a rotating wall as given by equations 13 to 16 may be determined from figure 15. The middle chart multiplies m by h, and the left and right charts multiply this product by the appropriate functions of ϕ. The arrows and broken lines indicate the use of the chart in an example to solve for the average lateral and vertical movements of the soil of height h for a given shear strain or deformation and the corresponding ϕ.

Critical slope of fill on soft layer.—Figure 16 may be used in solving equations 18 and 19. The three terms of equation 18 as rearranged on figure 16 are solved separately and must be added or subtracted as required. The lower-left chart is used for obtaining w_f/S_0 and $w_u\tan\phi$. The right-hand chart divides $2c$ by D. The small chart in the upper left is used for obtaining S_0, the slope of the triangle, when the equal area method is used.

In the illustrative example, b, H, S, w_f, w_u, ϕ, and D are given and the c required for equilibrium is to be found. Using method I, enter the upper-left chart with b/H and S to get S_0. The lower chart divides w_f by S_0 and multiplies w_u by $\tan\phi$. Next enter the right-hand chart with $2c/D$ (which equals $w_f/S_0 - w_u\tan\phi$), and with the given D find the required c. One may solve for S, if the other factors are known, by going through the chart in the reverse direction. If the slopes of the fill are assumed to be continued to form a triangle as in method II, $S = S_0$ and the upper-left chart may be disregarded.

Critical height of slopes.—The lower-left chart of figure 17 was constructed from Taylor's table of values of $\frac{c}{wH}$ for various values of slope angle and ϕ. The abscissas in the lower-left chart are $\frac{wH}{c}$ and the ordinates of the upper charts are wH. The curved lines in the lower-left chart are for circles through the toe of the slope when a more dangerous circle exists which passes below the toe.

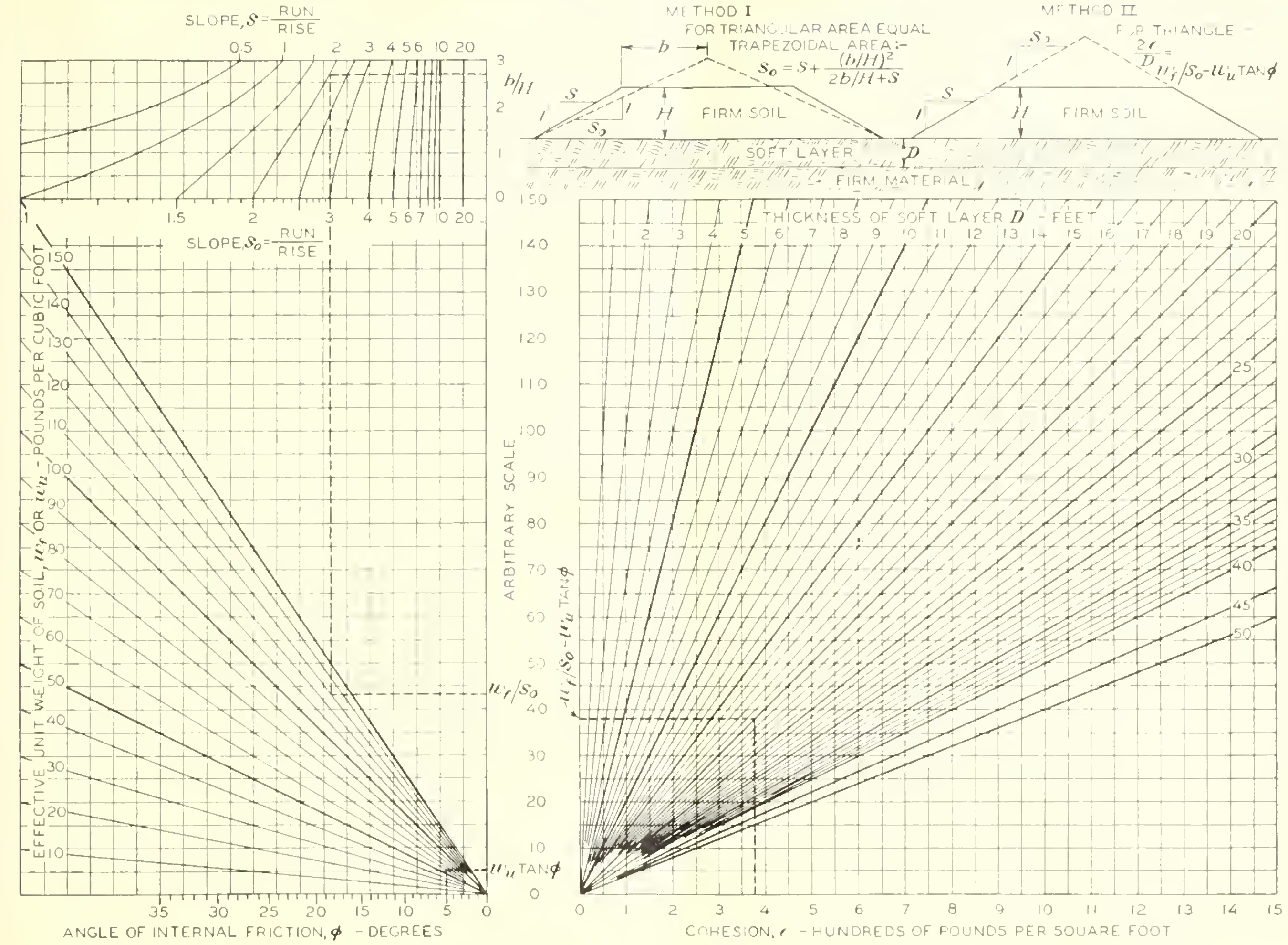
In the first example, find H when S is 1½:1, $\phi = 7.6°$, $c = 100$ pounds per square foot, and $w = 100$ pounds per cubic foot. By following the broken lines marked 1, H is found to be 10 feet. If H is 20 feet, one can go through the chart in reverse direction and find that the critical slope is 3.4:1.

The critical height of cohesionless materials is, according to this analysis, zero for slopes greater than ϕ and unlimited for slopes less than ϕ.

Greatest shear stress under long fill.—Equation 20 was solved for variously proportioned trapezoids and figure 18 was constructed for calculating the greatest shear stress produced by a given fill pressure. The left chart determines $\frac{s_G}{p}$ for given ratios of b/B. The right-hand chart multiples $\frac{s_G}{p}$ by p.

Cohesion required under uniform circular load.—Figure 19 may be used to determine F in equation 21. Values of s_G/p and p_1/p for substitution in the expression for F were taken from figure 10 for Poisson's ratio equals one-half. Broken lines on figure 19 represent values of F which were interpolated between values on the axis and values at the surface.

As an example of the use of figure 19, take $p = 8{,}000$ pounds per square foot, $r = 8$ inches, $w = 100$ pounds per cubic foot, and $\phi = 10°$. Then, to determine the required c at a depth of 16 inches, enter the chart at



BROKEN LINE IS FOR EXAMPLE :- TO SOLVE FOR c

METHOD I - FOR $b/H=2.7$ AND $S=2{:}1$, $S_0=3{:}1$ AND FOR $w_f=130$ LB. PER CU. FT., $w_f/S_0=43$, FOR $\phi=5°$ AND $w_u=60$ LB. PER CU. FT., $w_u\,\text{TAN}\,\phi=5$; THEN FOR $\frac{2c}{D}$ $(=43-5)=38$ AND $D=20$ FT, $c=380$ LB. PER SQ. FT.

METHOD II - TAKE $S_0=S$, THAT IS USE $S_0=2{:}1$ IN THE ABOVE EXAMPLE.

Figure 16.—Critical Slope of Fill on Soft Layer.

$z/r=16/8=2$, go over to the curve marked $\phi=10°$ and up to $F=0.11$. Substitute in equation 21 and obtain $c=8{,}000\times0.11-100\times16\times0.015=880-24=856$ pounds per square foot. $\frac{\text{Tan}\,\phi}{12}$ is used to change z from inches to feet.

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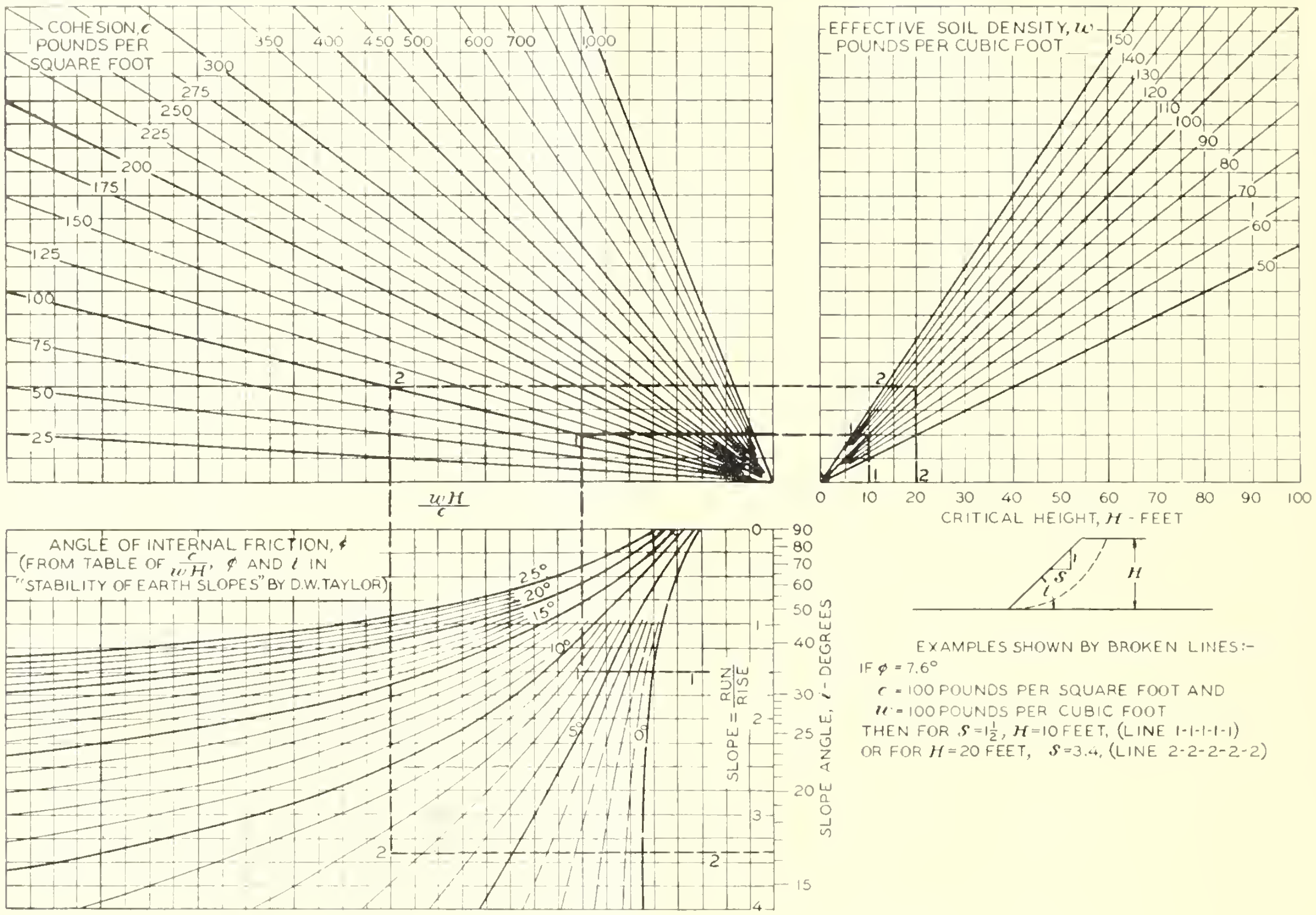

EXAMPLES SHOWN BY BROKEN LINES:-
IF ϕ = 7.6°
c = 100 POUNDS PER SQUARE FOOT AND
w = 100 POUNDS PER CUBIC FOOT
THEN FOR $S=1\frac{1}{2}$, H=10 FEET, (LINE 1-1-1-1-1)
OR FOR H=20 FEET, S=3.4, (LINE 2-2-2-2-2)

FIGURE 17.—CHART FOR CRITICAL HEIGHT OF SLOPES.

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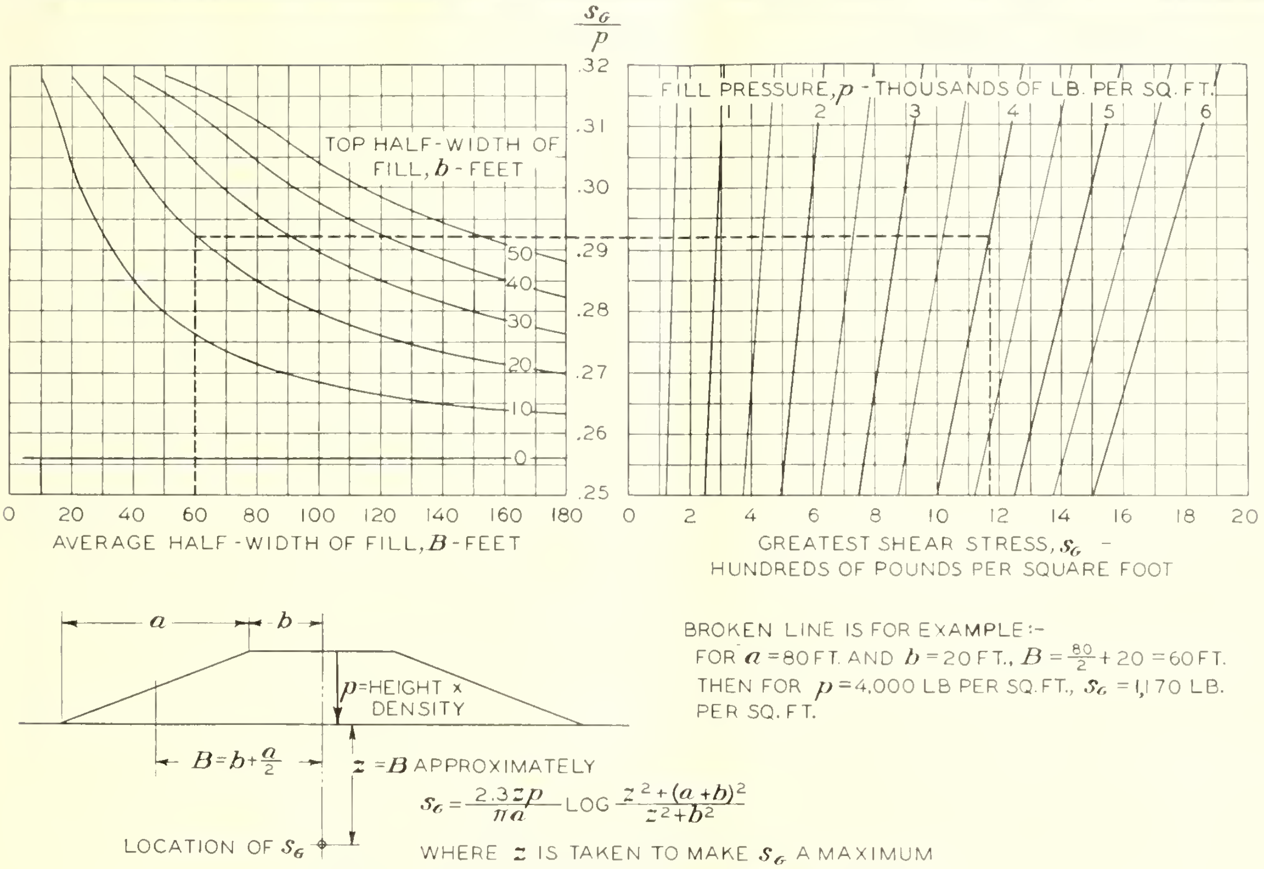
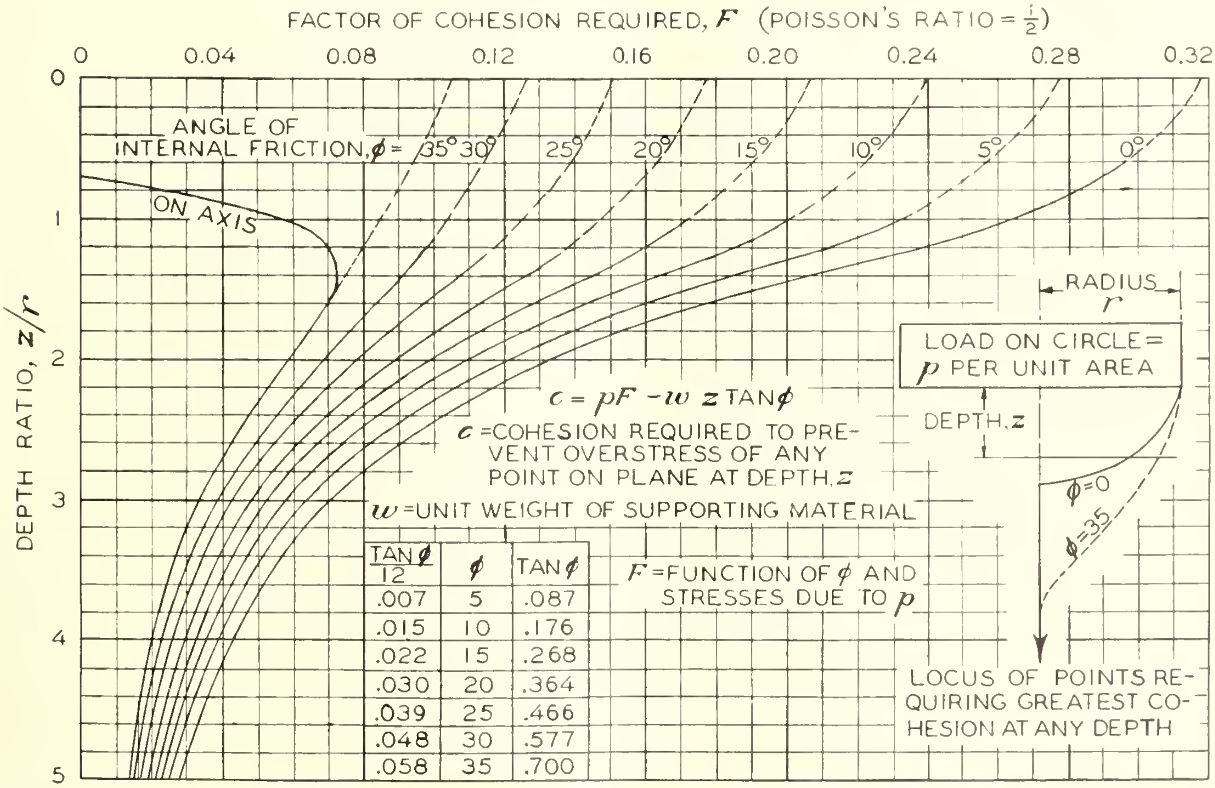


BROKEN LINE IS FOR EXAMPLE:-
FOR $a=80$ FT. AND $b=20$ FT., $B=\frac{80}{2}+20=60$ FT.
THEN FOR $p=4{,}000$ LB PER SQ. FT., $s_G=1{,}170$ LB. PER SQ. FT.

$$s_G=\frac{2.3zp}{\pi a}\text{ LOG }\frac{z^2+(a+b)^2}{z^2+b^2}$$

WHERE z IS TAKEN TO MAKE s_G A MAXIMUM

Figure 18.—Greatest Shear Stress Under Long Fill.



$c=pF-wz\text{ TAN}\phi$

c =COHESION REQUIRED TO PREVENT OVERSTRESS OF ANY POINT ON PLANE AT DEPTH z

w =UNIT WEIGHT OF SUPPORTING MATERIAL

F =FUNCTION OF ϕ AND STRESSES DUE TO p

$\frac{\text{TAN}\phi}{12}$	ϕ	TAN ϕ
.007	5	.087
.015	10	.176
.022	15	.268
.030	20	.364
.039	25	.466
.048	30	.577
.058	35	.700

Figure 19.—Graph of Factor for Determining Cohesion Required in Solid Supporting a Uniform Circular Load.

(Continued from page 146)

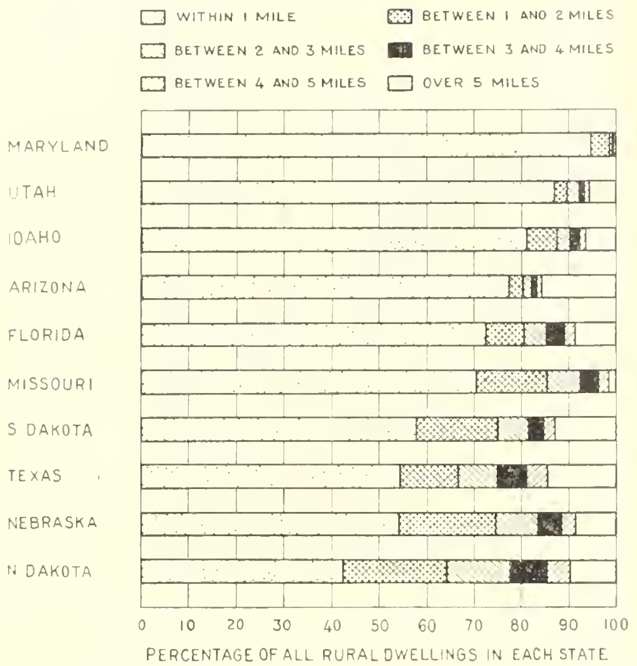


FIGURE 5.—PERCENTAGE OF ALL RURAL DWELLINGS WITHIN VARIOUS TRAVEL DISTANCES OF SURFACED ROADS IN EACH OF 10 STATES.

The percentage of all rural dwellings within 1 mile of a surfaced road ranged from 94.9 percent in Maryland, down to 42.2 percent in North Dakota.

Table 9 and figure 6 show, for the same 10 States, the percentage of all rural dwellings within various travel distances of either a surfaced road or a graded and drained road or, in other words, of an improved road. In these States, 80.9 percent of the rural dwellings were within 1 mile of an improved road. In Maryland, 99.1 percent of the rural dwellings were within 1 mile of an improved road and all of them were within 2 miles of an improved road. In Texas, on the other hand, the percentages of the rural dwellings within 1 mile and within 2 miles of improved roads were 62.0 percent and 73.9 percent, respectively. In all of the States for which the information was obtained, only a very small percentage of the rural residents need travel more than 1 or 2 miles from their homes to reach an improved road.

TABLE 8.—*Percentage of all rural dwellings within various travel distances of surfaced roads, in each of 10 States*

State	Within 1 mile	Within 2 miles	Within 3 miles	Within 4 miles	Within 5 miles
Arizona	77.1	80.4	82.1	83.1	84.2
Florida	72.6	80.5	85.6	89.1	91.4
Idaho	81.1	87.3	90.2	92.3	93.8
Maryland	94.9	98.9	99.7	99.9	100.0
Missouri	70.7	85.4	92.7	96.6	98.4
Nebraska	54.0	74.2	83.4	88.5	91.5
North Dakota	42.2	64.3	77.7	85.3	90.1
South Dakota	57.8	75.0	81.4	84.9	87.0
Texas	54.6	66.6	75.0	81.2	85.8
Utah	85.7	89.9	92.1	93.3	94.4
Average	65.0	77.5	84.4	88.7	91.5

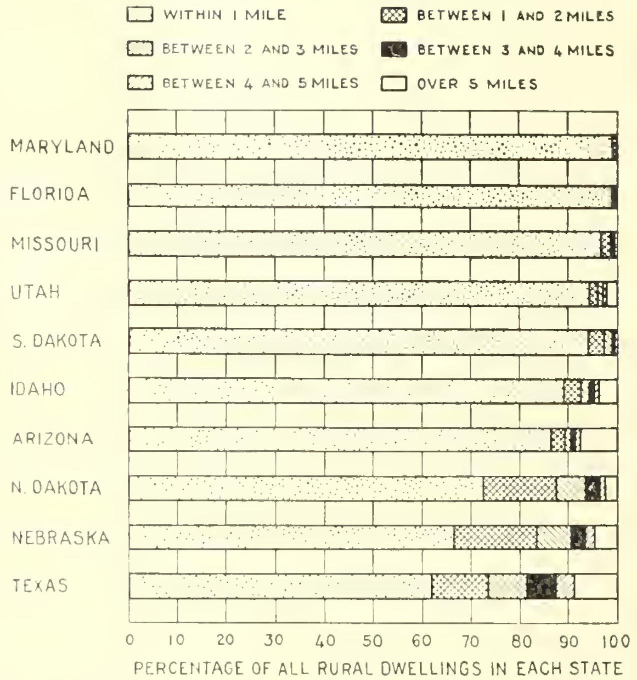


FIGURE 6.—PERCENTAGE OF ALL RURAL DWELLINGS WITHIN VARIOUS TRAVEL DISTANCES OF IMPROVED ROADS IN EACH OF 10 STATES.

In this report, all of the rural roads within a State have been considered as constituting a single system of roads. This is the concept of the average motorist since he generally does not know or care what administrative system or systems of roads he travels over in driving from one place to another. To those charged with the responsibility of financing, building, and maintaining roads, however, distinction between administrative systems is of the utmost importance. In nearly all States one group of public officials is responsible for State roads and other groups for county roads or township roads. The Federal Government has assumed responsibilities with respect to the 7 percent Federal-aid system, the 10 percent Federal-aid secondary system, and several systems or groups of roads serving national reservations, such as national forest highways, national forest development roads, national park roads, Indian roads, roads through public lands, and recently, roads of major importance from the standpoint of national defense. The sources and amounts of funds made available for each administrative system are responsibilities of legislative bodies. The segregation of road mileages into administrative systems is complicated because of overlapping juris-

TABLE 9.—*Percentage of all rural dwellings within various travel distances of improved roads, in each of 10 States* [1]

State	Within 1 mile	Within 2 miles	Within 3 miles	Within 4 miles	Within 5 miles
Arizona	86.7	89.5	90.9	91.8	92.4
Florida	99.1	99.6	99.7	99.8	99.8
Idaho	89.1	92.8	94.5	95.5	96.3
Maryland	99.1	100.0	100.0	100.0	100.0
Missouri	96.9	99.1	99.6	99.8	99.9
Nebraska	66.9	83.9	90.6	93.8	95.5
North Dakota	72.8	87.9	93.8	96.4	97.8
South Dakota	94.3	97.8	98.8	99.2	99.4
Texas	62.0	73.9	81.9	87.5	91.3
Utah	94.3	96.3	97.2	97.6	98.0
Average	80.9	88.2	92.1	94.5	96.1

[1] Improved roads include graded and drained roads and surfaced roads.

dictions. Also, significant comparisons of systems between States are difficult because of differences in the extent of the responsibilities of different jurisdictions. For example, in several States the State government assumes responsibility for all rural public roads, in others the responsibility is divided between the State and the counties, and in still others it is divided between the State, counties, and townships.

A complete and detailed analysis of roads by administrative system is being made in each State. For the reasons cited, such an analysis does not lend itself well to presentation on a national basis. Significant nationwide comparisons can be made, however, for the 7 percent Federal-aid system, for a group of the most important roads in each State designated as State highways in some and as primary State highways in others, and for all other rural roads regardless of jurisdiction which are mainly local roads in the sense that interest in them is not State-wide. Such comparisons will be made in a subsequent article.

REGULAR FEDERAL-AID FUNDS AUTHORIZED FOR 1942 AND 1943

The Federal Highway Act of 1940, which authorizes regular Federal-aid funds for highways, secondary or feeder roads, and grade crossings for the fiscal years 1942 and 1943, was approved on September 5, 1940. The act is in conformity with the congressional policy of authorizing in advance of the period for which they are available the Federal-aid funds for 2 years, enabling the various State legislatures, many of which meet biennially, to plan their highway budgets with foreknowledge of their approximate Federal-aid apportionments. Federal funds for other classes of road work are also provided by the act, the amounts provided for each fiscal year being as follows:

Item	Amount for each fiscal year
Federal-aid system	$100, 000, 000
Secondary or feeder roads	17, 500, 000
Elimination of hazards at grade crossings	20, 000, 000
National forest highways	7, 000, 000
National forest development roads	3, 000, 000
National park roads	4, 000, 000
Parkways	7, 500, 000
Public land roads	1, 500, 000
Indian roads	3, 000, 000

As in previous years, the Federal-aid highway and secondary road funds must be matched with State funds, and the grade crossing funds are outright grants to the States. Funds for these three classes of work for the fiscal year 1942 are required by law to be apportioned to the States, the District of Columbia, Hawaii, and Puerto Rico, by the Federal Works Administrator before next January 1. Formulas for apportioning the funds among the States remain unchanged.

Section 12 of the act specifically authorizes the Reconstruction Finance Corporation "to cooperate with States to finance, or to aid in financing, the acquisition of real property or interests in property * * * necessary or desirable for road projects eligible for Federal aid under the Federal Highway Act * * *." This provision will enable the long-term financing of highway rights-of-way through cities, thereby facilitating the early completion of necessary improvements that heretofore have not been undertaken because of the lack of sufficient current funds to pay both right-of-way and construction costs. High right-of-way costs, in many cases amounting to several times the actual construction costs, have retarded improvements to main routes through cities needed to eliminate traffic congestion and attendant danger and delay.

Section 19 of the act provides that: "In approving Federal-aid highway projects to be carried out with any unobligated funds apportioned to any State, the Commissioner of Public Roads may give priority of approval to, and expedite the construction of, projects that are recommended by the appropriate Federal defense agency as important to the national defense."

Under this provision of the law it should be possible to make an immediate beginning on the strategic highway program. A system of 75,000 miles of main highways has been selected by military and naval authorities as highly important for definite strategic reasons. Many sections of the system are already in satisfactory condition but there are also numerous substandard sections. Replacing weak bridges and widening and strengthening road surfaces and shoulders will be important parts of the work. The program is aimed at the elimination of critical weaknesses and restrictions on main highways.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF SEPTEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,705,324	$ 847,917	49.1	$ 4,642,001	$ 2,308,053	164.0	$ 1,837,290	$ 914,440	76.1	$ 2,166,993
Arizona	550,885	390,645	29.9	1,147,222	712,144	44.2	507,140	275,016	32.0	1,225,492
Arkansas	2,503,775	1,121,535	26.2	2,681,494	1,333,794	128.0	381,484	189,448	22.1	393,072
California	3,129,803	1,630,057	49.8	8,035,542	4,150,579	128.0	2,085,955	1,138,550	43.9	1,855,907
Colorado	1,171,028	658,169	108.0	2,227,256	1,237,574	114.6	513,621	289,477	77.6	2,173,268
Connecticut	15,000	7,500		2,706,448	1,320,856	21.9	264,250	124,573	4.7	1,052,590
Delaware	52,661	26,330	.9	1,969,981	984,211	31.6	155,407	51,044		1,048,125
Florida	732,053	366,026	20.1	3,232,131	1,605,786	96.8	840,971	420,485	17.4	2,313,581
Georgia	875,168	437,584	53.3	7,396,295	3,682,230	316.4	3,727,435	1,864,217	167.6	4,685,263
Idaho	518,918	317,895	52.6	1,393,325	856,177	122.3	378,325	162,646	17.7	1,581,336
Illinois	2,041,318	999,099	46.1	9,425,200	4,711,705	196.9	1,869,392	873,696	62.7	2,700,572
Indiana	2,242,510	1,121,255	58.9	6,537,321	3,118,504	112.8	3,461,808	1,698,749	52.1	884,241
Iowa	1,985,091	879,440	53.5	5,463,365	2,508,062	195.4	2,130,594	1,000,950	71.2	401,368
Kansas	601,220	300,610	38.4	7,516,257	3,757,784	449.6	2,866,854	1,433,426	256.3	3,236,143
Kentucky	1,233,766	615,327	19.0	3,948,134	1,974,067	108.0	661,649	330,825	53.5	2,893,632
Louisiana	643,730	316,311	5.4	12,720,814	3,405,451	64.7	1,170,213	577,418	27.3	2,863,497
Maine	750,620	374,022	15.8	1,068,408	534,203	29.5	243,710	121,855	5.0	604,532
Maryland	345,000	172,500	10.1	3,665,230	1,824,197	40.2	489,303	244,651	5.1	1,237,233
Massachusetts	553,156	276,153	4.7	3,672,471	1,827,381	31.0	177,871	88,935	.3	2,836,471
Michigan	1,085,862	542,931	37.9	11,593,150	5,708,474	334.2	1,623,316	794,758	50.1	288,080
Minnesota	2,067,822	983,419	173.7	7,522,270	3,748,500	491.9	803,941	401,971	29.5	3,155,788
Mississippi	763,732	305,416	30.5	7,013,274	3,141,143	335.3	1,943,060	948,730	112.5	1,454,294
Missouri	1,880,717	940,354	94.9	7,596,672	3,477,199	206.1	3,269,330	1,202,672	112.8	3,939,547
Montana	2,658,568	1,506,079	183.4	2,460,575	1,390,039	149.2	880,862	483,552	48.8	3,470,304
Nebraska	2,464,333	1,232,166	298.2	5,384,664	2,604,315	619.3	1,236,985	618,492	150.9	2,362,913
Nevada	1,196,352	1,029,857	60.9	1,374,094	1,194,076	62.1	5,600	4,881		759,186
New Hampshire	487,278	238,155	12.6	1,361,162	668,546	32.9	14,007	7,004		899,329
New Jersey	1,089,800	544,670	8.9	4,416,148	2,208,074	32.9	1,295,710	647,855	5.5	1,586,676
New Mexico	1,133,944	700,925	89.6	1,480,152	900,375	88.3	622,420	375,195	38.5	1,124,863
New York	2,008,626	992,285	41.7	16,742,726	8,187,377	242.3	4,096,017	1,519,684	46.0	925,195
North Carolina	2,782,105	1,390,055	130.5	3,891,121	1,946,132	187.3	2,261,000	1,102,970	104.1	1,321,066
North Dakota	1,055,750	565,838	101.5	2,684,408	1,509,122	211.0	3,169,464	1,621,950	269.1	3,197,318
Ohio	1,805,773	902,886	23.8	13,637,102	6,794,222	124.4	2,953,710	1,476,580	24.9	4,281,168
Oklahoma	866,292	459,550	50.0	3,486,033	1,843,453	112.9	1,395,085	686,614	50.9	4,092,767
Oregon	1,513,189	909,630	108.9	2,724,902	1,619,495	56.4	1,311,660	602,750	30.2	871,464
Pennsylvania	1,977,624	961,473	26.8	12,841,490	6,377,960	127.4	3,564,299	1,774,179	29.2	1,868,526
Rhode Island	292,212	145,890	2.5	1,457,124	727,160	15.1	300,411	150,180	1.9	852,533
South Carolina	518,801	241,980	36.5	2,605,266	1,261,281	176.5	933,689	416,210	69.7	2,139,230
South Dakota	1,557,814	881,564	235.7	4,338,480	2,567,480	582.6	1,239,140	749,990	225.2	2,448,861
Tennessee	524,608	262,304	10.4	3,646,358	1,823,179	106.6	1,547,042	773,521	61.6	3,708,269
Texas	3,321,834	1,636,523	231.2	7,788,195	3,861,187	364.4	3,965,998	1,878,790	172.2	6,033,154
Utah	412,082	300,075	32.5	1,237,235	918,263	61.8	677,430	276,330	26.3	733,481
Vermont	214,830	107,415	11.1	1,499,414	748,058	40.7	320,628	160,314	5.7	263,842
Virginia	1,288,200	597,453	41.1	2,556,897	1,236,647	62.5	2,117,568	1,040,995	27.1	1,204,406
Washington	1,678,909	884,294	25.5	2,990,562	1,582,235	69.1	776,697	413,400	12.4	754,323
West Virginia	507,150	252,395	16.6	3,531,880	1,757,741	107.3	666,530	333,265	12.9	1,671,200
Wisconsin	3,142,906	1,533,610	94.9	3,791,451	1,883,380	158.3	325,553	153,950	14.4	3,407,486
Wyoming	1,290,783	814,104	142.2	1,003,016	638,987	108.1	536,608	342,993	65.8	649,481
District of Columbia	154,300	77,150	1.2	383,942	191,971	3.3	250,100	117,016	1.5	325,209
Hawaii	29,184	11,010		354,190	179,072	5.2	546,461	273,046	9.2	1,498,907
Puerto Rico	20,338	9,980	.3	1,597,615	790,710	26.0	106,769	52,580	1.9	717,500
TOTALS	63,442,744	32,819,811	2,997.3	234,440,463	115,338,611	7,697.3	68,520,362	33,202,818	2,803.4	98,159,682

U S GOVERNMENT PRINTING OFFICE 1940

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF SEPTEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 175,144	$ 87,363	9.4	$ 1,065,679	$ 531,120	48.6	$ 257,167	$ 128,233	12.2	$ 354,133
Arizona	58,228	21,068	4.7	282,532	203,973	13.9				234,685
Arkansas	331,054	154,443	7.7	280,505	140,041	27.8	101,825	68,376	6.7	72,639
California	341,885	185,938	10.5	722,425	390,814	31.4	108,742	40,601	.7	808,868
Colorado				199,428	89,535	1.9	71,801	40,467	3.0	160,220
Connecticut	52,818	25,299	.8	317,216	154,114	3.8	102,386	48,738	1.8	165,663
Delaware	69,537	34,768	7.8	98,995	41,350	7.7				268,125
Florida	12,030	6,015		408,106	204,053	.3	537,863	243,458	19.8	175,237
Georgia	38,761	19,381	5.5	633,812	301,906	59.2	506,881	253,441	42.7	1,039,418
Idaho	27,402	16,902	6.0	88,521	54,574	13.0	208,304	95,931	10.3	136,439
Illinois	724,335	361,655	46.4	1,496,050	731,363	41.5	451,400	225,700	20.8	210,877
Indiana	346,670	172,131	23.0	195,102	97,516	11.7	91,936	45,900	4.4	916,997
Iowa	1,179,975	561,025	201.4	1,050,588	499,676	243.1	681,738	322,680	218.2	159,685
Kansas	221,992	110,996	17.5	728,915	367,366	45.0	485,580	242,789	45.9	1,109,693
Kentucky	506,671	148,835	40.9	636,726	214,195	35.0	388,174	95,751	18.0	284,327
Louisiana	41,637	20,819	3.7	256,292	128,091	22.0				458,796
Maine	115,010	56,716	7.6	189,314	89,248	9.7	29,000	14,500	1.3	8,024
Maryland				177,996	88,998	8.2				427,378
Massachusetts	110,536	54,886	2.7	521,271	258,386	10.7				498,612
Michigan	186,548	93,274	22.1	1,456,329	731,554	113.2	244,200	122,100	18.5	511,141
Minnesota	190,545	91,761	16.7	753,335	375,951	118.6	568,703	284,352	73.4	845,959
Mississippi	172,962	86,481	10.6	671,452	330,226	31.4	414,400	195,665	19.8	424,732
Missouri	206,794	103,397	29.9	500,342	249,798	47.9	434,949	139,947	53.8	668,229
Montana	458,768	259,458	63.7	249,798	140,960	19.2	22,269	12,566	1.2	644,158
Nebraska	328,695	164,188	53.7	660,960	324,496	89.0	143,879	71,940	24.4	231,413
Nevada	151,328	130,622	33.6	126,614	104,928	12.9	40,504	35,310	2.1	55,950
New Hampshire	48,387	23,241	2.2	97,444	46,738	1.2				167,812
New Jersey	319,500	159,750	10.6	264,090	131,965	9.0	61,333	30,660	2.4	499,449
New Mexico	101,564	63,386	13.1	396,987	244,848	14.7	273,612	100,232	14.1	85,190
New York	981,201	490,601	40.8	2,065,875	997,860	54.2	366,640	149,143	11.4	72,269
North Carolina	472,550	236,275	48.0	627,203	314,568	49.7	23,680	10,500	.8	317,257
North Dakota	39,433	21,136	.3	145,151	79,399	1.1	27,520	14,750	2.4	1,014,688
Ohio	413,957	206,913	14.6	2,808,570	1,403,025	91.8	271,880	135,940	7.1	741,852
Oklahoma	536,288	284,638	37.9	254,865	134,374	19.2	112,680	35,269	3.1	945,470
Oregon	276,538	147,840	43.6	281,370	125,830	23.0	101,801	58,990	11.3	207,154
Pennsylvania	755,242	371,516	30.6	1,572,441	785,312	39.3	42,000	21,000	.3	214,836
Rhode Island	107,556	53,749	1.8	130,339	65,143	1.8				91,158
South Carolina	92,800	43,750	8.9	633,123	236,366	72.2	188,647	80,250	50.0	193,595
South Dakota				3,624	3,624					1,280,913
Tennessee	105,038	52,519	7.9	48,612	24,306	3.0	186,188	93,094	7.2	909,350
Texas	983,983	483,876	138.4	665,963	328,010	70.6	392,750	181,475	26.5	845,653
Utah	3,767	2,500		240,082	151,040	25.9	31,200	23,000	5.7	121,152
Vermont	139,900	48,305	5.4	366,408	103,750	14.0				22,500
Virginia	168,540	80,212	9.0	460,344	211,971	25.1	112,476	46,810	1.2	286,029
Washington	159,951	83,988	5.8	462,792	245,539	25.2	277,321	144,800	25.0	119,534
West Virginia	154,950	76,950	9.6	155,669	77,834	7.1	79,808	39,904	4.4	446,556
Wisconsin	180,214	89,751	2.2	758,070	378,910	27.7				649,380
Wyoming	389,348	229,898	39.0	114,117	72,162	7.9				117,160
District of Columbia				130,684	64,842	1.4				22,150
Hawaii	170,361	85,037	6.4	108,585	54,841	2.2				158,778
Puerto Rico				302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	12,650,393	6,303,252	1,102.0	26,862,936	13,274,129	1,668.0	8,496,425	3,921,402	774.0	20,481,691

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF SEPTEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,100	$ 4,100			1	$ 717,904	$ 697,827	6			$ 16,800	$ 16,800			5	$ 943,965
Arizona	363,787	356,072	4			7,592	7,592				6,006	6,006			2	242,091
Arkansas	77,452	77,452	1		5	1,090,995	1,086,848	7	1		210,067	209,848	5		11	430,330
California	181,075	181,075	2		3	886,145	707,664	3	1		26,464	26,464			12	1,688,531
Colorado						283,222	283,222	1			3,401	3,401			1	919,512
Connecticut	153,792	148,343		1		633,988	628,437	7								449,334
Delaware	45,759	45,759			9	137,499	137,499	1		7	13,839	13,839	4		1	467,605
Florida	203,348	198,849	2		1	37,626	37,626	1			127,859	127,334		1	15	1,290,976
Georgia	98,753	98,753	2		11	765,788	765,788	9	5	1	607,118	607,118	4	3	5	1,792,357
Idaho	104,842	104,237	1			102,897	100,072	4			85,358	85,358			33	424,756
Illinois	431,062	377,451	3	1	16	2,374,331	2,171,855	9	1	42	235,161	200,862	1		65	2,028,650
Indiana	237,674	237,674	1	1	28	892,262	892,262	6	1	43	86,166	86,166			27	1,002,549
Iowa	160,688	151,600	1		29	394,280	316,258	4		44	96,576	90,456	1		24	1,217,969
Kansas	426,425	426,425	5		5	531,945	531,467	7	1	1	205,254	205,254	3	1	18	1,089,153
Kentucky	161,777	161,777	4		10	1,042,369	1,042,369	11			9,457	9,457			3	567,149
Louisiana	95,496	95,496	1			345,122	291,627	2			627,885	570,153	11			804,609
Maine	149,214	149,214	1	1	4	26,841	26,841		1	1	128,350	128,350	1		1	251,016
Maryland	180,996	180,993	1		2	472,109	440,316	2	2	5	15,600	15,600			3	771,815
Massachusetts	15,710	15,710				336,591	326,168	1			1,034	1,034			1	2,024,008
Michigan	682,519	682,519	4	1	13	1,466,427	1,466,427	5	4	2	197,698	197,698		1	22	822,730
Minnesota	515,366	507,509	4	1	2	1,472,970	1,472,970	12	3	8	244,644	244,644	1	3	8	1,010,984
Mississippi	23,760	23,760				494,634	494,634	7	1		166,500	166,500	3		3	741,349
Missouri	150,832	150,832		2		1,843,554	1,555,174	7	3		1,057,603	829,073	4	2	1	973,697
Montana	276,720	276,720	5		2	154,873	85,232				99,948	96,681	1		2	410,875
Nebraska	195,478	195,478		1	5	737,924	737,924	10	1	5	132,930	132,930	4		1	441,925
Nevada						115,732	115,732	2		5	42,138	42,138			20	115,071
New Hampshire	100,989	100,953	3			82,452	82,452	1			82,358	81,538	2		1	309,842
New Jersey						772,219	772,219	5			75,490	75,490		1	4	1,207,397
New Mexico	140,504	140,504	1		6	110,443	110,443	1			189,836	164,453	3			482,206
New York	307,110	306,000	2	1		3,875,479	3,807,413	10	18		283,405	279,305		2	8	2,824,415
North Carolina	112,535	112,535	1	1	7	1,007,343	1,007,103	13	3	6	94,050	94,050		2	18	852,601
North Dakota	432,190	432,190	5		11	103,750	103,750	2		6	132,270	108,473	1		6	741,485
Ohio	369,098	347,336	4		2	2,462,415	2,413,225	10	3	13	654,157	628,380	3		2	2,397,021
Oklahoma	245,738	244,771	2	1	22	594,442	593,095	10		2	200,773	197,357	1		41	1,890,452
Oregon						462,847	315,518	4			5,790	5,790			2	357,935
Pennsylvania	528,871	528,871	4	1		1,563,370	1,553,894	14	1		1,446,614	1,446,409	9	2		3,359,718
Rhode Island	3,831	3,750				191,039	191,039		1							95,913
South Carolina	120,394	120,394	2	2	6	365,705	365,705	3	1	8	175,150	174,748			33	927,058
South Dakota	72,600	72,600	1	1	2	281,762	280,902	7		2	462,535	446,585	10			740,832
Tennessee	204,265	204,265	1	2		48,523	48,523	1			159,203	159,203		2		1,715,788
Texas	959,152	957,100	8		3	1,367,684	1,354,563	12	1	6	70,450	70,450	1		2	2,006,230
Utah	9,569	9,358			2	40,813	40,813			11	76,367	76,260	2		27	276,632
Vermont	2,969	2,969			1	209,428	209,428		2	1	34,546	34,546			7	223,562
Virginia	81,101	80,765	1	2	3	216,368	215,038	3			608,649	608,649	3	2	8	601,096
Washington	199,253	199,253	2	1	4	166,673	165,173	2	1	1	135,862	135,862	1	1	5	449,464
West Virginia						229,982	229,982	2			8,220	8,220			3	1,141,717
Wisconsin	743,581	728,501	6	2	7	541,103	512,062	3	2	3	17,167	17,167			5	1,358,033
Wyoming						540,923	540,923	5								193,577
District of Columbia						59,061	59,061	1								150,010
Hawaii						194,036	194,028	2			9,494	9,494			2	292,509
Puerto Rico						584,007	579,336	11								414,857
TOTALS	9,570,435	9,439,913	85	23	222	33,437,487	32,165,519	246	58	223	9,366,242	8,935,593	79	23	458	47,933,356

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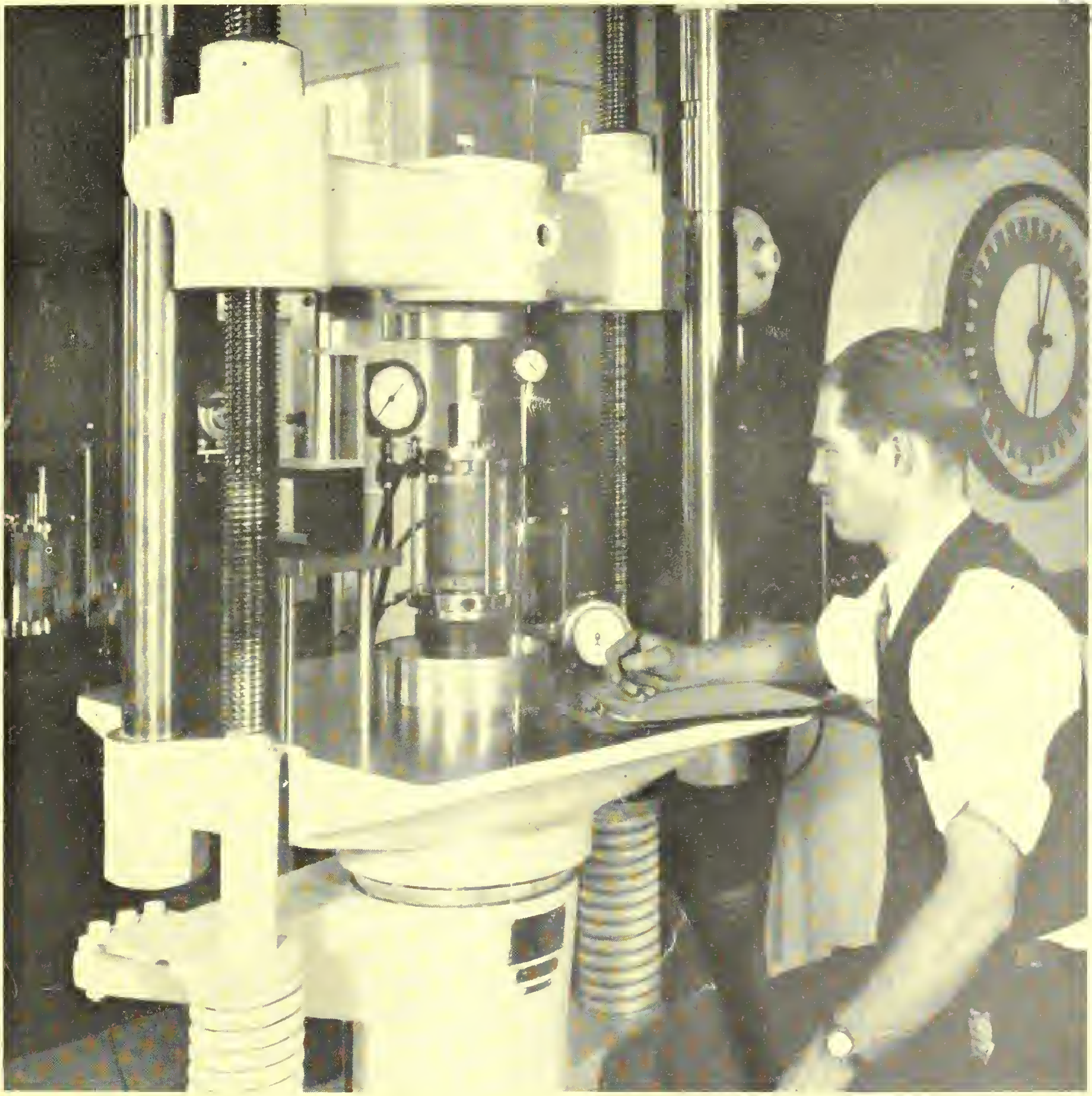

LABORATORY DETERMINATION OF SOIL STABILITY

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PUBLIC ROADS

A Journal of Highway Research

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 9

November 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE SETTLEMENT OF EARTH EMBANKMENTS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTATION

Reported by **L. A. PALMER**, Associate Research Specialist, and **E. S. BARBER**, Junior Highway Engineer

ONE of the most important considerations in the design of highway embankments is the amount of settlement that may be expected. Methods of estimating settlement caused by soil consolidation due solely to loss of water from compressible foundation soils have already been described.[1] The present report is chiefly concerned with settlement resulting from lateral displacement of soil, designated as S_L, as distinguished from settlement caused by consolidation, designated as S_C.

The question of ultimate supporting power has been considered at length in three previous publications [2] [3] [4] and the analyses are made with reference to ultimate fill loads that will cause the supporting earth to fail completely, that is, deform without any definite limit. Obviously, however, more information is required for the satisfactory design of embankments.

For example, it is entirely possible for a high fill to subside several feet due to displacement in the supporting soil but without the occurrence of failure in that supporting soil. Assumption in the design that a large factor of safety against ultimate failure will assure one that displacement by lateral yield will be a small quantity may not accomplish the desired result. Hogentogler and Allen [5] have pointed out the fallacy of such an assumption and have proposed the use of values, c' and ϕ', which are certain percentages of the unit cohesion c, and the angle of internal friction ϕ, respectively, which appear in formulas for computing the supporting power of the earth below the fill. This procedure is based on the use of the complete shearing stress-deformation relation instead of ultimate values.

PLASTIC YIELD ASSUMED INSTEAD OF ELASTIC DEFORMATION

In continuation of this study, it is proposed to make use of a principle presented by A. Nadai [6] called the "stationary flow of a plastic mass." This principle leads to the development and use of formulas that are similar to the expressions for Hooke's law for elastic bodies but which differ from Hooke's law in that plastic yield is assumed in the place of elastic deformation.

The basic principle involved in this report is a simple one and is that a certain earth movement, too small to be comparable to displacements characteristic of failure of the earth itself, may ruin a structure. More specifically, the fill load may be much too small to cause failure of the supporting earth, yet it may be large enough to be disastrous to the highway. In brief, it is necessary to estimate S_L. For this purpose two things are needed: A ratio of stress to deformation, herein designated as C, the modulus of deformation, which has no reference to the nature of the deformation, whether it be elastic or plastic deformation or both; and a value of Poisson's ratio, μ.

Experimental work has indicated that the value of μ for compressible types of soil may vary from 0.35 for soils containing much air to 0.50 for soils that are saturated with water.

For the cases in which μ is less than $\frac{1}{2}$, it is very likely that the small soil samples undergo some degree of volume diminution owing to the escape of air or water during the stabilometer test. It cannot, however, be concluded that the same volume change occurring in such tests can take place as readily and quickly in large earth masses. Therefore, it has been customary to take μ as $\frac{1}{2}$ in computing S_L in large earth masses even though the laboratory value of μ is not this quantity, and this value of μ is used in computing S_L in this report.

Figure 1 illustrates the essential features of the laboratory stabilometer used in the study of stresses within a cylindrical soil sample. The stabilometer is often referred to as the "triaxial shear test device." Several types of this device and the principles governing their use have been described by Hogentogler and Barber.[7]

In stabilometer tests, cylindrical soil samples, encased in rubber sleeves, are compressed to complete failure by the application of vertical load. During the loading there may be no lateral pressure on the specimen (a simple compression test in this case) or a variable or a constant lateral pressure may be applied from start to finish of the test. The test may be made with or without porous stones at the flat ends of the cylindrical samples.

The vertical load is applied through the plunger by means of a hydraulic testing machine. At the beginning of a test, the head of the machine is lowered until contact is made with the plunger, as shown in the cover illustration, and the platen and head of the machine remain fixed in position until loading is begun. A definite fluid pressure is then applied to the sample, figure 1, through the inlet valve. Since the machine is fixed in position the lateral fluid pressure tends to push a saturated sample upward against the plunger with a variable force depending on the mobility of the sample. For a saturated soft soil, the vertical and lateral pressures may become equal under these initial conditions. When the sample contains much air the fluid pressure tends to compress the air or cause its escape thus shortening the sample without increasing its diameter.

---

[1] The Theory of Soil Consolidation and Testing of Foundation Soils, by L. A. Palmer and E. S. Barber. PUBLIC ROADS, vol. 18, No. 1, March 1937.

[2] Principles of Soil Mechanics Involved in Fill Construction, by L. A. Palmer and E. S. Barber. Proceedings of the Highway Research Board, Seventeenth Annual Meeting, Dec. 1937.

[3] Principles of Soil Mechanics Involved in the Design of Retaining Walls and Bridge Abutments, by L. A. Palmer, PUBLIC ROADS, vol. 19, No. 10, Dec. 1938.

[4] Design of a Fill Supported by Clay Underlaid by Rock, L. A. Palmer. PUBLIC ROADS, vol. 20, No. 8. Oct. 1939.

[5] Important Considerations in Soil Mechanics, by C. A. Hogentogler and Harold Allen, Bulletin, American Society for Testing Materials, No. 94, Oct., 1938.

[6] Plasticity (chapter 14, pp. 75 to 79, incl.), A. Nadai. Engineering Societies Monographs, McGraw-Hill Book Company, Inc., first edition, 1931.

[7] Essential Features of Triaxial Shear Tests, by C. A. Hogentogler and E. S. Barber. PUBLIC ROADS, vol. 20, No. 7, Sept., 1939.

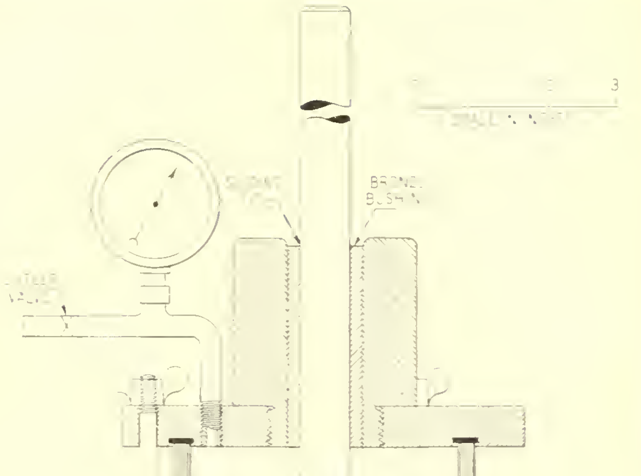

Figure 1.—Stabilometer of the Plunger Type.

Vertical load is then increased by elevating the platen at a constant rate of 0.05 inch per minute. An automatic recording device gives the complete vertical load versus change in height curve for the entire test. Any change from the initial height, h, is designated as Δh.

For the plotted data shown in this paper, the soil cylinders were 1.95 inches in diameter and the height, h, was 4 inches. Porous stones, sometimes placed at the two flat ends of the sample, were not used in these tests.

MODULUS C DETERMINABLE FROM STABILOMETER TEST DATA

During a quick stabilometer test, a relatively impermeable and saturated soil undergoes deformation without appreciable volume change. If, however, the soil contains air, loading tends to compress the air, according to Boyle's law, with consequent reductions of volume and height of the sample.

The known decrease in height due either to compression of air or its escape is not considered as deformation in computing the modulus of deformation, C. In computing this modulus, it is assumed that the soil deforms at constant volume in which case μ, Poisson's ratio, is $\frac{1}{2}$.

Reference is made to figure 2, which illustrates three distinctly different types of soil behavior during the early period of loading in the stabilometer or triaxial compression device. In figure 2–A, with a lateral pressure of 6,000 pounds per square foot, the developed vertical pressure with the machine in fixed position was also 6,000 pounds per square foot. The point, $l=v=6{,}000$ therefore falls on the axis of v, vertical pressure, which is the axis of zero $\frac{\Delta h}{h}$. Figure 2–B illustrates another type of behavior. Here the developed v with the machine in fixed position was 5,600 pounds per square foot with l, the lateral pressure, equal to 6,000. The point A for $l=v=6{,}000$ is not on the $\frac{\Delta h}{h}=0$ axis but corresponds to a value of 0.0004 for $\frac{\Delta h}{h}$. Figure 2–C illustrates still another type of behavior. Here the developed v with the machine in fixed position was 5,200 for $l=6{,}000$ and under this system of initial stresses, the height h decreased until the value of $\frac{\Delta h}{h}$ became 0.0075. Then as load was applied and v was increased, the sample began to deform. At the point A for $v=l=6{,}000$, $\frac{\Delta h}{h}$ is seen to be 0.0089.

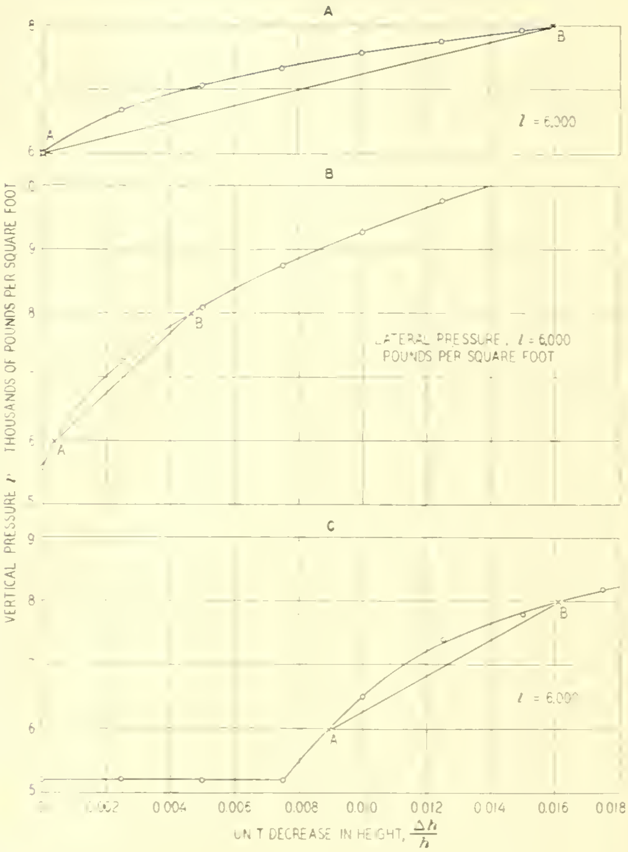

Figure 2.—Stress-Strain Curves From Triaxial Compression Tests.

Figure 2–A is characteristic of a relatively soft saturated soil of low permeability; figure 2–B is characteristic of a relatively stiff saturated soil of low permeability; and figure 2–C is characteristic of a stiff soil that contains an appreciable volume of air.

The horizontal distance from the point A, where $l=v$, to the $\frac{\Delta h}{h}=0$ axis is zero for curve A in figure 2, 0.0004 for curve B, and 0.0089 for curve C. These variable distances denote reductions in h due to volume changes without distortion and therefore represent small consolidations of the sample that are usually unavoidable, appear in their entirety, and are completely accounted for in consolidation tests. Thus the horizontal distances from A to the $\frac{\Delta h}{h}=0$ axis properly fall in the category of S_C settlement. The change in $\frac{\Delta h}{h}$ beyond the point A is indicative of distortion without volume change (S_L settlement).

The modulus of deformation, C, is taken as a secant modulus in this paper and is the slope of the secant line drawn from the initial point A, figure 2, where $l=v$, to another point B, determinable from the conditions of the particular problem. Since there are an infinite number of points beyond A, there may be an infinite number of secant lines and of moduli C. There is but one secant line, however, for a specific problem fixing the point B. Thus the nature of the problem may be such that the value of $v-l$ at the point B is 2,000 pounds per square foot. This is illustrated in figure 2. If $l=6{,}000$ and $v-l=2{,}000$, then $v=8{,}000$, the ordinate value of the point B, figure 3. In figure 2–A, the slope of AB is $\frac{8{,}000-6{,}000}{0.016-0}=125{,}000$ pounds per square foot $=C$. In figure 2–B, the slope of AB is $\frac{8{,}000-6{,}000}{0.0047-0.0004}=465{,}000$ pounds per square foot $=C$. In figure 2–C, the slope of AB is $\frac{8{,}000-6{,}000}{0.0161-0.0089}=$ 278,000 pounds per square foot $=C$, rounding the value of C to the nearest 1,000 pounds per square foot.

For any vertical load greater than that corresponding to the initial point, A, figure 2, it is necessary to correct for the changed horizontal cross-sectional area of the deformed sample in computing v. This correction is contained in the following expression for v:

$$v=\frac{P}{A}\left(1-\frac{\Delta h}{h}\right)+l \quad \text{-------------------------} \quad (1)$$

where P is the net load, equal to the load on the plunger plus the weight of the plunger minus the product of the lateral pressure, l, and the cross-sectional area, a, of the plunger stem. A is the initial cross-sectional area of the sample.

SETTLEMENT OF FILL UNDER ITS OWN WEIGHT CONSIDERED

Figure 3 illustrates a symmetrical earth fill with equal slopes. The Y direction is the direction of the length of the fill which is perpendicular to the plane of the figure. If the fill is long in comparison with its width, displacement in the Y direction is zero. The Z direction is the vertical one and OZ (fig. 3) is the axis of symmetry of any vertical cross section that is assumed to be of unit thickness in the Y direction. In the plane of the diagram (fig. 3) the X direction is horizontal. The directions are indicated by the coordinates, X, Y, and Z. The origin, 0, is at the base of the fill and on the vertical axis of symmetry. The letters x, y, and z denote both direction and distance whereas X, Y, and Z denote direction only. The normal stresses p_x and p_z act in the X and Z directions, respectively, and their values at any point depend upon the coordinates x and z of the point. The shearing stress, s_{zx}, acts in the X direction and in a plane to which the Z direction is perpendicular. The shearing stress, s_{xz}, acts in the Z direction and in a plane to which the X direction is perpendicular. For a condition of equilibrium, $s_{xz}=s_{zx}$.

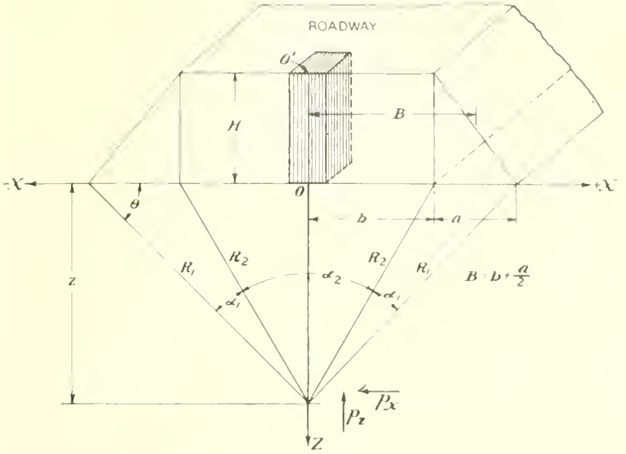

FIGURE 3. DIAGRAM OF SYMMETRICAL EARTH FILL.

The fill problem, figure 3, involves two dimensions, that is, only soil movements in the X and Z directions are of concern. The letter V designates the displacement of a particle at a point (x, z) in the Z direction and U designates the displacement of the particle at the same point in the X direction. The rate of change of V with respect to z, when x is constant, is $\frac{\partial V}{\partial z}$ and denotes the strain in the Z direction. The rate of change of U with respect to x, at constant z, is $\frac{\partial U}{\partial x}$, the strain in the X direction. The strain in the Y direction is zero. The problem is one of plane strain or deformation. From Hooke's law and Nadai's principle,

$$\frac{\partial V}{\partial z}=\frac{1}{C}[p_z-\mu(p_x+p_y)] \quad \text{--------------} \quad (2)$$

$$\text{strain in } Y \text{ direction}=0=\frac{1}{C}[p_y-\mu(p_x+p_z)] \quad \text{--------} \quad (3)$$

and

$$\frac{\partial U}{\partial x}=\frac{1}{C}[p_x-\mu(p_y+p_z)] \quad \text{------------} \quad (4)$$

Nadai refers to C as a "constant" rather than a modulus and, for a material that undergoes distortion without change in volume, takes μ as 1/2.

Then, according to Nadai's principle, applicable to flow in a plastic mass, equation 2 would become

$$\frac{\partial V}{\partial z}=\frac{1}{C}[p_z-1/2(p_x+p_y)] \quad \text{-----------} \quad (5)$$

Consider the column of earth, 00′ on the vertical axis of symmetry in figure 3. For the present, consider the origin as moved from 0 to 0′. Any vertical distance, z, is then considered as directed downward from the roadway which is the horizontal plane containing 0′. Let w denote the weight per cubic foot of fill material assumed to be homogeneous. Then at any depth z in the column 0′0, $p_z=wz$ and $p_x=K'wz$ where K' is the ratio of lateral to vertical pressure at the depth z. From equation 3

$$p_y=\mu(p_x+p_z)=\mu(K'wz+wz)$$

Substituting this value for p_y in equation 2

$$\frac{\partial V}{\partial z}=\frac{1}{C}\Big[wz-\mu(K'wz+\mu K'wz+\mu wz)\Big]$$

or

$$\frac{\partial V}{\partial z}=\frac{wz}{C}\left[1-\mu K'-\mu^2(K'+1)\right] \text{------} (6)$$

Integrating equation 6, assuming K' constant,

$$V=\frac{wz^2}{2C}\left[1-\mu K'-\mu^2(K'+1)\right]+f(x) \text{------} (7)$$

On the axis of symmetry, $f(x)$, a function of x alone, becomes a constant, K_1, and for points on this axis,

$$V=\frac{wz^2}{2C}\left[1-\mu K'-\mu^2(K'+1)\right]+K_1 \text{------} (8)$$

If it is assumed that $V=0$ at $z=H$, a condition which will exist if the undersoil is unyielding and there is only the settlement S_L of the fill to consider, K_1 may then be evaluated so that equation 8 becomes

$$V=\frac{w}{2C}(z^2-H^2)[1-\mu K'-\mu^2(K'+1)] \text{------} (9)$$

and for $\mu=1/2$,

$$V=\frac{3w}{8C}(1-K')(z^2-H^2) \text{------} (10)$$

Here V denotes the downward displacement of a soil particle on the axis of symmetry and at any depth z from the roadway. The greatest vertical displacement is at $z=0$ at $0'$. At this point,

$$V=S_L=-\frac{3w}{8C}H^2(1-K') \text{------} (11)$$

The use of equation 11 is limited by the fact that there is no sure way of determining K'. Theoretically, its value is greater than 0 and less than 1. If $K'=0$ is taken, then S_L is a maximum value and on the side of safety.

SETTLEMENT S_L OF THE UNDERSOIL DETERMINED

The origin is now taken at the point 0, figure 3. The angles, α_1 and α_2 of figure 3 are expressed in radians and from the diagram it is evident that

$$2\alpha_1+\alpha_2=2 \text{ arc cot } \frac{z}{a+b}$$

and

$$\alpha_1=\text{arc cot } \frac{z}{a+b}-\text{arc cot } \frac{z}{b}.$$

It has been shown [2] that on the axis of symmetry, OZ,

$$p_z=\frac{p}{\pi}\left(2\alpha_1+\alpha_2+\frac{2b}{a}\alpha_1\right) \text{------} (12)$$

and

$$p_x=\frac{p}{\pi}\left(2\alpha_1+\alpha_2+\frac{2b}{a}\alpha_1-\frac{4z}{a}\log_e\frac{R_1}{R_2}\right) \text{------} (13)$$

where p_x and p_z are normal stresses (at any depth z on OZ) due solely to the fill load.

From equation 3, $p_y=\mu(p_x+p_z)$ and substituting this value for p_y in equation 2,

$$\frac{\partial V}{\partial z}=\frac{1}{C}[p_z-\mu(p_x+\mu p_x+\mu p_z)] \text{------} (14)$$

By substituting equations 12 and 13 in 14 one obtains

$$\frac{\partial V}{\partial z}=\frac{p}{\pi C}\left[(1-\mu-2\mu^2)\left(2 \text{ arc cot } \frac{z}{a+b}+\frac{2b}{a}\text{ arc cot } \frac{z}{a+b}-\frac{2b}{a}\text{ arc cot } \frac{z}{b}\right)+\mu(1+\mu)\frac{4z}{a}\log_e\frac{\sqrt{(a+b)^2+z^2}}{\sqrt{b^2+z^2}}\right] (15)$$

It must be remembered that on integrating equation 15, it is desired to know V on the axis of symmetry and hence, $x=0$. The last term of equation 15 is the only one giving difficulty in integration. This may be integrated by parts and by the transformation,

$$z=(a+b)\tan\theta$$

where θ is the angle shown in figure 3.

The vertical displacement, V, of a soil particle at the point 0, figure 3, and S_L, are identical in magnitude. In general, however, V refers to the vertical displacement of a particle at any point and not just at 0. Hence, it is a special value of V, namely its value at 0, the sum of all the strains from 0 to a given depth z, that is equal to S_L.

By, integration of equation 15 between the limits, 0 and z, one obtains

$$\begin{aligned}V \text{ at } 0=\frac{p}{\pi C}\Bigg\{&(1-\mu-2\mu^2)\left(2+\frac{2b}{a}\right)\\&\left[z \text{ arc cot } \frac{z}{a+b}+\frac{a+b}{2}\log_e\left(1+\frac{z^2}{(a+b)^2}\right)\right]\\&-(1-\mu-2\mu^2)\left(\frac{2b}{a}\right)\left[z \text{ arc cot } \frac{z}{b}+\frac{b}{2}\log_e\left(1+\frac{z^2}{b^2}\right)\right]\\&+\frac{4\mu(1+\mu)}{a}\left[\frac{z^2}{2}\log_e\frac{\sqrt{z^2+(a+b)^2}}{\sqrt{z^2+b^2}}\right.\\&\left.+\frac{(a+b)^2}{2}\log_e\frac{\sqrt{z^2+(a+b)^2}}{a+b}-\frac{b^2}{2}\log_e\frac{\sqrt{z^2+b^2}}{b}\right]\Bigg\}\end{aligned} \text{------} (16)$$

S_L denotes the diminution in thickness of a given depth of undersoil due to lateral displacement, and for $\mu=1/2$ equation 16 reduces to

$$\begin{aligned}S_L=V \text{ at } 0=\frac{3p}{\pi Ca}&\left[\frac{z^2}{2}\log_e\frac{\sqrt{z^2+(a+b)^2}}{\sqrt{z^2+b^2}}\right.\\&\left.+\frac{(a+b)^2}{2}\log_e\frac{\sqrt{z^2+(a+b)^2}}{a+b}-\frac{b^2}{2}\log_e\frac{\sqrt{z^2+b^2}}{b}\right]\end{aligned} \text{------} (17)$$

It is convenient to obtain solutions of equations 12 and 16 by graphical methods. With reference to figure 3, let $B=b+\frac{a}{2}$. Then for $a=0$, the trapezoidal load becomes a uniform strip load and $\frac{b}{B}=1$. For $b=0$, the load diagram becomes triangular and $\frac{b}{B}=0$. All of the possible symmetrical trapezoidal load diagrams are then contained within the limiting cases, $\frac{b}{B}=0$ and $\frac{b}{B}=1$. Now write for equation 12,

$$p_z=pf$$

where $f=\frac{1}{\pi}\left(2\alpha_1+\alpha_2+\frac{2b}{a}\alpha_1\right)$.

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[2] See footnote 2, p. 161.

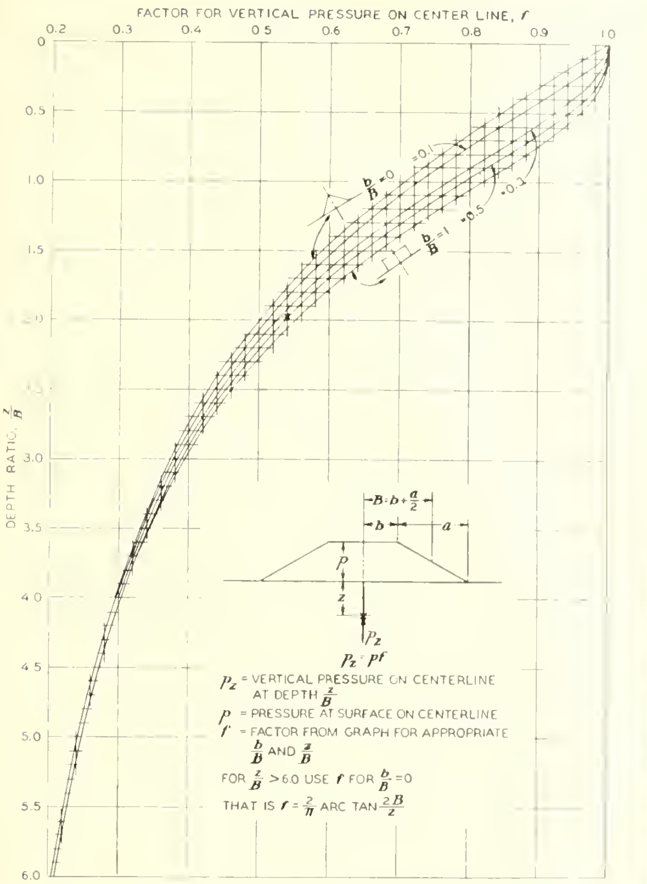

FIGURE 4.—GRAPH OF VERTICAL PRESSURE ON CENTERLINE UNDER A SYMMETRICAL FILL.

The value of f depends on the depth z of any point on the centerline and on the distances, a and b (fig. 3). The curves in figure 4 are obtained by plotting values of f against the "depth ratio," $\frac{z}{B}$. The five curves of this figure are for the ratios of $\frac{b}{B}$ equal to 0, 0.1, 0.3, 0.5, and 1.0. For ratios that are intermediate, a value for f corresponding to a given $\frac{z}{B}$ ratio may be obtained by interpolation. For depths such that $\frac{z}{B}$ exceeds 6.0, the value for f is taken from the formula for $\frac{b}{B}=0$, regardless of what the actual value of $\frac{b}{B}$ may be since there is practically complete coincidence of all the curves at $\frac{z}{B}=6.0$.

In fills having the same B and the same height, H, the vertical cross-sectional area is the same. Hence, it is convenient to use the value B as a basis for constructing charts such as figure 4 for use in computations.

Equation 16 may be written

$$S_L=\frac{p}{C}\left(b+\frac{a}{2}\right)F=\frac{pB}{C}F \qquad (18)$$

where

$$F=\frac{1+\mu}{\pi(1-b/B)}\Bigg\{(1-\mu)\left[(2-b/B)^2\log_e\sqrt{1+\frac{(z/B)^2}{(2-b/B)^2}}-(b/B)^2\log_e\sqrt{1+\frac{(z/B)^2}{(b/B)^2}}\right]+\mu(z/B)^2\log_e\sqrt{\frac{(z/B)^2+(2-b/B)^2}{(z/B)^2+(b/B)^2}}+(1-2\mu)z/B\left[(2-b/B)\text{ arc cot }\frac{z/B}{2-b/B}-(b/B)\text{ arc cot }\frac{z/B}{b/B}\right]\Bigg\}$$

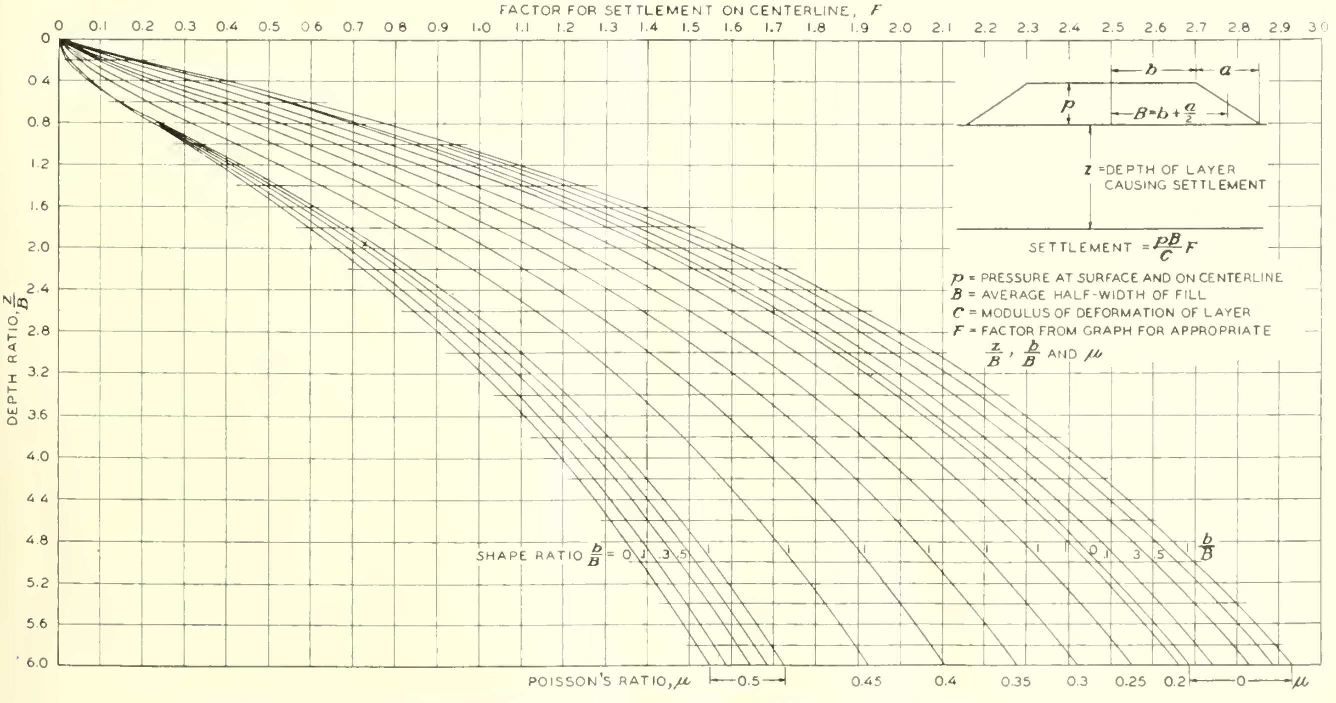

FIGURE 5.—GRAPH OF SETTLEMENT ON CENTERLINE UNDER A SYMMETRICAL FILL.

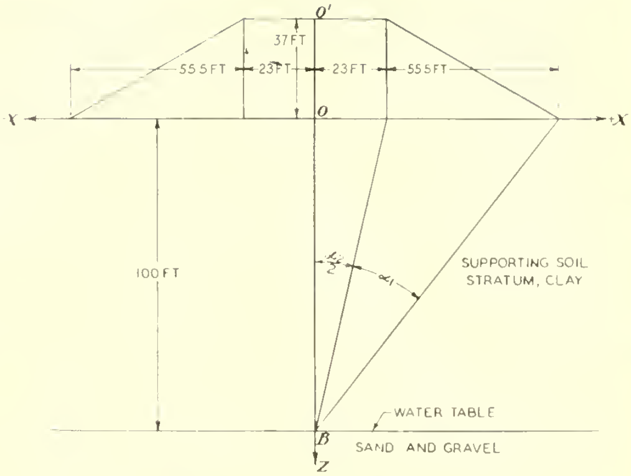

Figure 6.—Symmetrical Fill Supported by Clay.

The value of F in equation 18 depends on μ, Poisson's ratio. The curves of figure 5 were constructed for values of μ varying from 0 to ½ by plotting values of $\frac{z}{B}$ against F for different values of $\frac{b}{B}$. A numerical example will be given illustrating the use of the chart, figure 5.

COMPUTATION OF S_L ILLUSTRATED

Consider the fill, figure 6, of height, $H=37$ feet, with a 1½ to 1 slope and supported by a clay stratum 100 feet thick which is underlaid by sand and gravel. The supporting soil and fill material have different properties. The average weight per cubic foot of the compacted fill material is $w=126$ pounds. The width of the roadway is 46 feet so that b (fig. 3) is 23 feet. The distance, a, is $1\frac{1}{2}\times 37=55.5$ feet. The value of $B=b+\frac{a}{2}=23+\frac{55.5}{2}=50.8$ feet and the ratio, $\frac{b}{B}$, is $\frac{23}{50.8}=0.453$. The value of p is $wH=37\times 126=4{,}662$ pounds per square foot. The thickness of the deformable clay layer is 100 feet and the depth ratio, $\frac{z}{B}$, is $\frac{100}{50.8}=1.97$.

In previous publications [2 3] it has been shown that the greatest shearing stress anywhere in the undersoil below a symmetrical fill is $0.32\ p$. This greatest shearing stress is located at a distance equal approximately to B below the point 0, figure 3, and is equal to $\frac{1}{2}\ (v-l)$ where v is the vertical pressure and l the lateral pressure at the point of greatest shearing stress.

Then since $\frac{1}{2}\ (v-l)=0.32\ p$, $v-l=0.64\ p$ and theoretically this is the greatest $v-l$ value that exists anywhere in the supporting earth below the embankment. For $p=4{,}662$ pounds per square foot, the greatest value of $v-l$ is $0.64\times 4{,}662=2{,}984$ pounds per square foot. With reference now to figure 7 which shows the v versus $\frac{\Delta h}{h}$ characteristics of the undersoil for $l=0$ and $l=4{,}180$ pounds per square foot, the point B is determined for each curve. For $v-l=2{,}984$, when $l=0$, $v=2{,}984$ and for $l=4{,}180$, $v=2{,}984+4{,}180=7{,}164$ pounds per square foot. The ordinate values of B are then 2,984 for $l=0$ and 7,164 for $l=4{,}180$. The value of $\frac{\Delta h}{h}$ at the initial point A for $l=0$ is -0.0001. The initial point A for $l=4{,}180$ is at that point of the upper curve where $v=l=4{,}180$. For $l=4{,}180$, the slope of AB, upper curve of figure 2, is $\frac{7{,}164-4{,}180}{0.0310-0.0036}=109{,}000$ pounds per square foot $=C$. For $l=0$, the slope of AB,

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[2 3] See footnotes 2 and 3, p. 161.

(Continued on page 172)

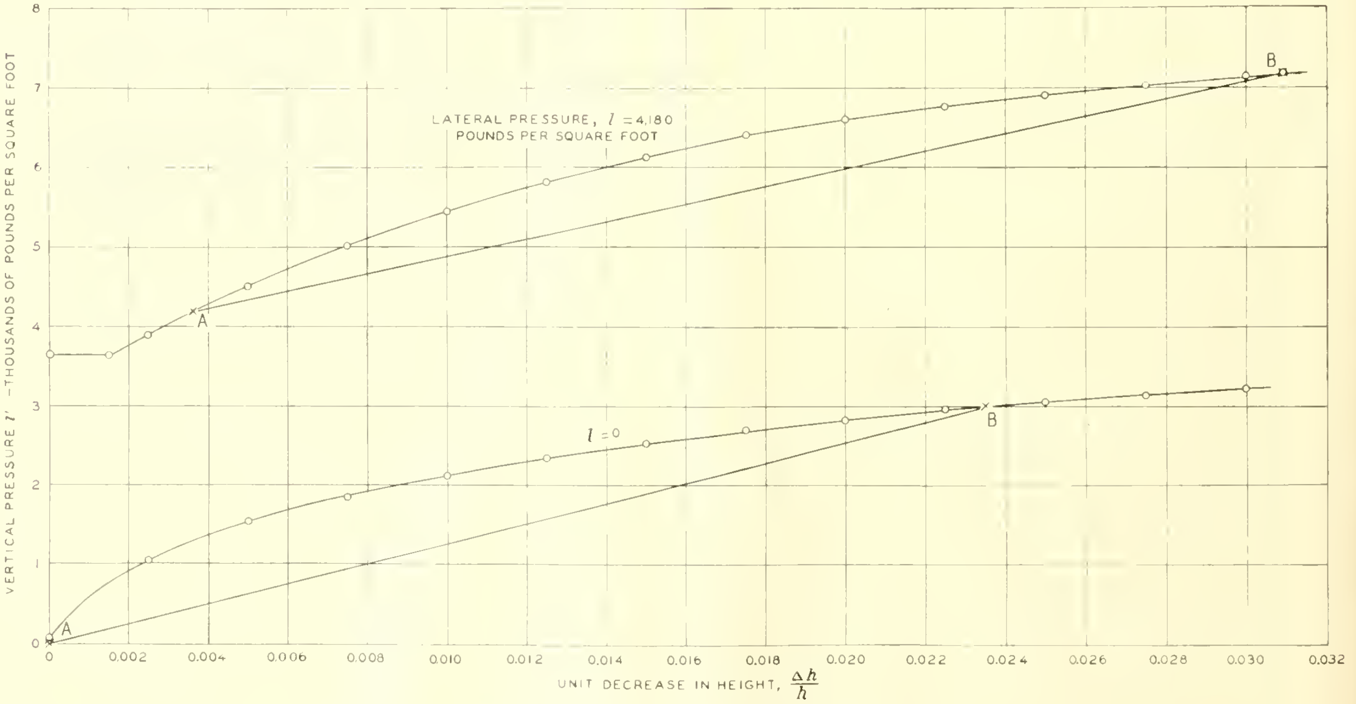

Figure 7.—Triaxial Test Data for Foundation Soil.

HORIZONTAL FORCES AND MOMENTS ON BRIDGE ABUTMENTS AND RETAINING WALLS

EFFECT OF CONCENTRATED MOVING LIVE LOADS ON THE BACKFILL

Reported by **E. I. FIESENHEISER**, Assistant Bridge Engineer, District 4, Public Roads Administration

THE PURPOSE of this paper is to present methods of computing the overturning effect of live loads by taking into account the distribution of stresses through the soil. The paper presents:

(1) Exact equations for the force and moment based on the theoretical variation of stress.
(2) Simplified approximate equations sufficiently accurate for most cases.
(3) A determination of the position of the live load for maximum moment.
(4) Charts for use in computing.

In designing bridge abutments and retaining walls the horizontal pressure of the backfill material on the wall must be taken into account. The amount of this pressure and its overturning moment are usually determined by assuming that the pressure increases uniformly with the depth of fill. This assumption results in either a triangular or trapezoidal pressure diagram.

In dealing with the problem of truck wheels or concentrated live loads on top of the backfill or roadway it is customary to assume an added depth of fill or surcharge. The loads are assumed to be distributed over an area and the depth of surcharge is taken as the depth of a volume of backfill material equal to the loads in weight.

The moment effect of the live loads upon the wall has been a matter for conjecture and the method of adding a surcharge has doubtless been used for lack of a better or more precise method. However, recent experiment and progress in the field of soil mechanics have pointed the way to a very different solution of this problem.

STRESS DISTRIBUTION EQUATIONS DEVELOPED

When taking into account the stress distribution in the soil reference is usually made to the work of Joseph Boussinesq, a noted elastician, who derived equations for the stresses in the interior of an elastic solid. The conditions assumed by Boussinesq are a semi-infinite, elastic, isotropic solid bounded by a plane surface upon which a single concentrated load acts in a direction perpendicular to the plane surface. Published equations,[1] give the normal and shearing components of stress in the interior of the solid. However, these stress components are only those acting on planes parallel to the surface plane.

Assuming the horizontal surface of the backfill to be the boundary plane and keeping in mind that the plane of the wall or abutment is vertical, an expression is needed for the horizontal component of stress acting upon a plane perpendicular to the surface plane. Such an expression in terms of rectangular coordinates with the origin at the load point is $\sigma_x=\frac{3P}{2\pi}\frac{x^2z}{(x^2+y^2+z^2)^{5/2}}$, for the stress in the x-direction, the z-axis being vertical and the x- and y-axes horizontal. This is a simplified expression derived by assuming the term Poisson's ratio equal to one-half, which is in accordance with a fundamental principle of soil mechanics that the soil particles are incompressible or incapable of a change in volume. Equations from which it is derived may be found in texts on the theory of elasticity.[2]

Actual working conditions for which the above expression would be applicable would be a soil of perfect elasticity with elastic properties the same in all directions (conditions of an isotropic soil). Some soils may very nearly approach these conditions, while others may be far from elastic and isotropic. However, with regard to retaining walls, experiments have indicated that the shape of the pressure variation curve is similar to that obtained from the theoretical expression and that the quantitative differences may be taken into account by the use of empirical constants dependent upon the type of soil.

In experiments on retaining walls conducted at the Iowa Engineering Experiment Station, Ames, Iowa, by M. G. Spangler[3] the horizontal pressure due to concentrated surface loads was measured. By transforming the theoretical expression an empirical equation was devised which gives pressure intensities corresponding to the measured intensities. The equation devised is $h_c=\frac{kP}{x^n}\frac{x^2z}{(x^2+y^2+z^2)^{5/2}}$ in which k is a constant depending upon the type of soil and n is a constant depending upon the relative rigidity of the wall and backfill. Obviously more experimenting is needed with different types of soil before values of these constants can be determined for all cases, since these experiments were made with gravel only.

HORIZONTAL FORCES AND OVERTURNING MOMENTS COMPUTED

In applying the equation to the computation of horizontal forces and overturning moments working values of k equal to 1.3 and n equal to 1/4 will be used, assuming a gravel backfill and conditions similar to those in the experiments conducted by Spangler. If values for k and n can be determined for other materials and conditions, such values may be substituted in the equations to be derived. Forces and moments will be directly proportional to the constant k and inversely to x^n, x being the distance from the load to the wall. (See fig. 1.)

As shown in figure 1, with the origin directly under the load, the coordinates of a point on the wall are x, y, and z, where

x = horizontal distance from the load to the wall.
y = horizontal distance from the load parallel to the wall.
z = vertical distance below the top of the fill.
$R=\sqrt{x^2+y^2+z^2}$.
P = the concentrated load.

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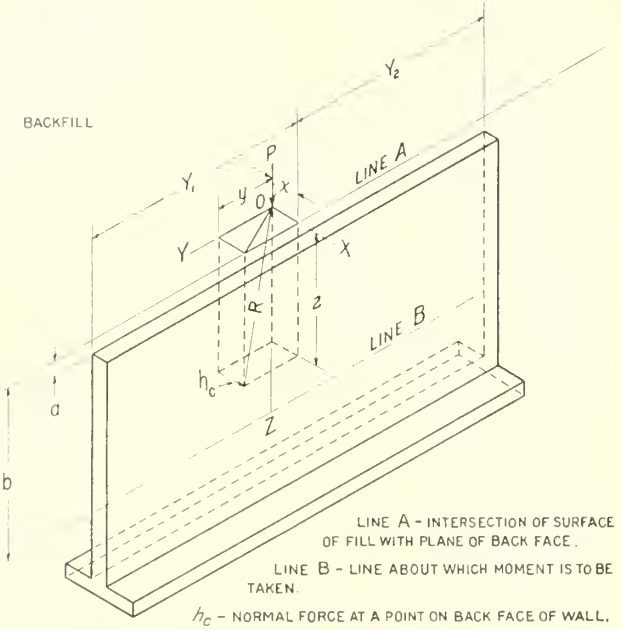
[1] Application des Potentieis a L'etude de L'equilibre et du Mouvement des Solides Elastique, by M. J. Boussinesq. Gauthier-Villars. Paris. 1885. p. 104.

[2] For the solution in terms of cylindrical coordinates see Theory of Elasticity, by S. Timoshenko. McGraw Hill, New York. 1934. pp. 328-331.

[3] Horizontal Pressures on Retaining Walls Due to Concentrated Surface Loads, by M. G. Spangler. Iowa Engineering Experiment Station Bulletin 140. 1938.



LINE A - INTERSECTION OF SURFACE OF FILL WITH PLANE OF BACK FACE.

LINE B - LINE ABOUT WHICH MOMENT IS TO BE TAKEN.

h_c - NORMAL FORCE AT A POINT ON BACK FACE OF WALL.

FIGURE 1.—DIAGRAM SHOWING RETAINING WALL AND LOAD ON BACKFILL.

a = vertical distance from top of fill to top of wall.
b = vertical distance from top of fill to a horizontal line where the moment is desired.
M = moment at the line b distance below top of fill.
H = total horizontal force acting on the wall above the line.
h_c = intensity of horizontal pressure at any point on the wall.

To get a clear picture of the stress variation expressed by the equation, curves may be plotted holding x and y constant while z varies (see fig. 2) and holding x and z constant while y varies (see fig. 3).

In dealing with a bridge abutment attention is called to the fact that the top or surface of the backfill is not always at the top of the wall. A part of the earth pressure may be taken by the floor slab or diaphragms at the end of the bridge and transferred as a thrust through the superstructure. In such a case only the horizontal force acting on the area of the wall unit is of interest. Accordingly a distance a from the surface of the fill to the top of the wall is introduced to take care of this condition.

Referring to figure 2, $h_c=\frac{kP}{x^n}\cdot\frac{x^2z}{(x^2+y^2+z^2)^{5/2}}$

The differential area of wall $dA=dy\ dz$ and the force

$$dH=h_cdA=\frac{kP}{x^n}\cdot\frac{x^2zdydz}{(x^2+y^2+z^2)^{5/2}}$$

Taking as z-limits the distances b and a and as y-limits the distances Y_1 and Y_2,

$$H=kPx^{(2-n)}\int_a^b\int_{Y_2}^{Y_1}\frac{zdydz}{(x^2+y^2+z^2)^{5/2}}\text{-------}(1)$$

Integrating in z-direction and substituting limits,

$$H=\frac{kPx^{(2-n)}}{3}\left\{\int_{Y_2}^{Y_1}(a^2+x^2+y^2)^{-\frac{3}{2}}dy-\int_{Y_2}^{Y_1}(b^2+x^2+y^2)^{-\frac{3}{2}}dy\right\}$$

Integrating this expression in the y-direction and substituting limits,

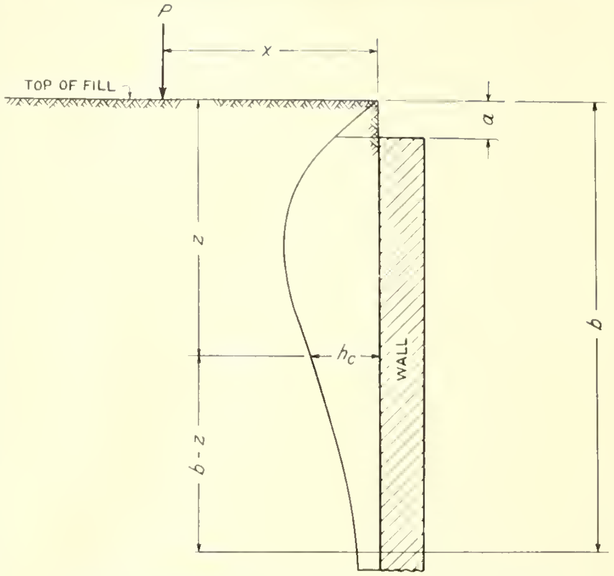


FIGURE 2.—STRESS VARIATION, x AND y CONSTANT.

$$H=\frac{kPx^{(2-n)}}{3}$$

$$\left\{Y_1\left[\frac{1}{(a^2+x^2)\sqrt{a^2+x^2+Y_1^2}}-\frac{1}{(b^2+x^2)\sqrt{b^2+x^2+Y_1^2}}\right]\right.$$

$$\left.-Y_2\left[\frac{1}{(a^2+x^2)\sqrt{a^2+x^2+Y_2^2}}-\frac{1}{(b^2+x^2)\sqrt{b^2+x^2+Y_2^2}}\right]\right\}\text{--}(2)$$

In deriving an equation for the overturning moment the differential of moment about a line b distance below the top of fill is

$$dM=\frac{kP}{x^n}\frac{x^2z(b-z)dydz}{(x^2+y^2+z^2)^{5/2}}$$

and the total moment

$$M=\frac{kPx^2}{x^n}\int_a^b\int_{Y_2}^{Y_1}\frac{z(b-z)dydz}{(x^2+y^2+z^2)^{5/2}}\text{-------}(3)$$

Integrating first in the z-direction and substituting the limits b and a,

$$M=\frac{kPx^{(2-n)}}{3}\int_{Y_2}^{Y_1}\frac{1}{(x^2+y^2)}$$

$$\left[\frac{a^3+bx^2+by^2}{(a^2+x^2+y^2)^{3/2}}-\frac{b}{(b^2+x^2+y^2)^{1/2}}\right]dy$$

Integrating now in the y-direction, substituting the limits Y_1 and Y_2, and simplifying, the total moment becomes

$$M=\frac{kPx^{(2-n)}}{3}[(a^3+bx^2)A+b(C-B)]\text{-------}(4)$$

in which

$$A=\frac{1}{a^2}\left(\frac{\alpha}{ax}-\frac{\gamma}{a^2+x^2}\right)\text{-------}(5)$$

$$B=\frac{\phi}{bx}\text{-------}(6)$$

$$C=\frac{1}{a^2}\left(\gamma-\frac{x\alpha}{a}\right)\text{-------}(7)$$

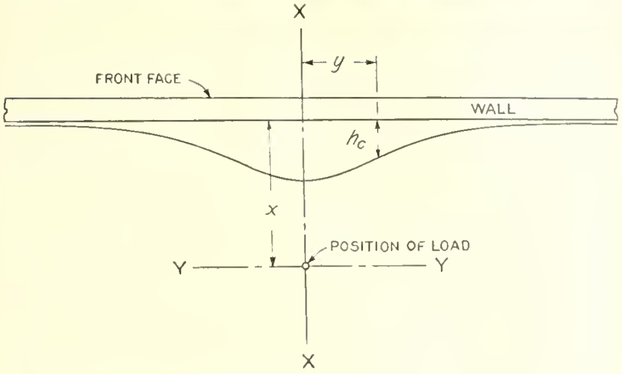


FIGURE 3.—STRESS VARIATION, x AND z CONSTANT.

and

$$\gamma=\frac{Y_1}{\sqrt{a^2+x^2+Y_1^2}}-\frac{Y_2}{\sqrt{a^2+x^2+Y_2^2}} \qquad (8)$$

$$\alpha=\tan^{-1}\left[\frac{a}{x}\frac{Y_1}{\sqrt{a^2+x^2+Y_1^2}}\right]-\tan^{-1}\left[\frac{a}{x}\frac{Y_2}{\sqrt{a^2+x^2+Y_2^2}}\right] \qquad (9)$$

$$\phi=\tan^{-1}\left[\frac{b}{x}\frac{Y_1}{\sqrt{b^2+x^2+Y_1^2}}\right]-\tan^{-1}\left[\frac{b}{x}\frac{Y_2}{\sqrt{b^2+x^2+Y_2^2}}\right] \qquad (10)$$

Referring to equations 8, 9, and 10 it should be noted that Y_1 and Y_2 are distances measured parallel to the wall from the load to the ends of the wall. When the load is between the ends, which will ordinarily be the case, Y_1 will be assumed positive and Y_2 negative. The result will be the addition of the two terms in obtaining the value of the angles α and ϕ and the term γ. Angles α and ϕ are expressed in radians. When computing these angles if a conversion table is not at hand the angles may first be found in degrees and decimals of a degree from tables of natural functions, then multiplied by the constant 0.017453 to change degrees to radians.

For the particular case when the top of the fill coincides with the top of the wall the term a becomes zero. To obtain the moment the original expression, equation 3, may be integrated direct between limits b and 0. An alternate method is to substitute zero for a in the final equations 4 to 10. In the latter case the result will be zero divided by zero for the terms A and C. These indeterminate forms may be evaluated in the usual manner by differentiating. Taking third derivatives of numerators and denominators the values of the fractions can be found. By either method the resulting equation will be:

For $a=0$,

$$M=\frac{kPx^{(2-n)}}{3}\cdot b(D-B) \qquad (11)$$

in which the term

$$D=\frac{Y_1}{x^2\sqrt{x^2+Y_1^2}}-\frac{Y_2}{x^2\sqrt{x^2+Y_2^2}} \qquad (12)$$

EQUATIONS APPLIED TO BRIDGE ABUTMENT PROBLEM

Equations 2 and 4 are expressions for the force and its overturning moment caused by a single load P concentrated at a point and placed at x distance from the wall. If the familiar principle of superposition is applied when there are more loads than one to deal with, the effects of the loads may be added together and the total moment will be the sum of the moments produced by the individual loads.

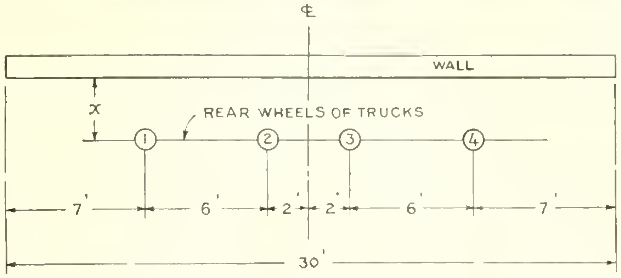


FIGURE 4.—PLAN OF ABUTMENT WALL, SHOWING POSITION OF REAR WHEELS OF 2 TRUCKS.

In applying the above equations to a bridge abutment a structure carrying two 10-foot traffic lanes is assumed. An abutment wall at least 30 feet long will ordinarily be required to support the two-lane superstructure. Such an arrangement of rear truck wheels as that indicated in figure 4 may be assumed.

Using the 30-foot wall as a basis for computing, a large number of solutions were worked out for the moment using different values for b and a. These computations revealed the fact that for relatively small values of a or x there is not much difference in the effect produced by wheels 1 and 4 and that produced by wheels 2 and 3. For example this difference was found to be less than 1 percent for the values $b=30$ feet and $a=0.5$ foot. In dealing with a wall of this length or greater it will usually be sufficiently accurate to compute the moment for a single load and multiply the result by the number of loads to obtain the total effect.

It can be seen that a single computation of force and moment by the above equations involves considerable work. Fortunately it is possible to derive much simpler equations which will be sufficiently accurate for the average wall. In a problem of this kind unnecessary refinements in calculation cannot be justified, especially if values of the soil constants cannot be determined with great exactness. The accuracy of the final result will depend upon experimental values of k and n. If infinite limits are assumed for y, letting $Y_1=+\infty$ and $Y_2=-\infty$, the following expressions are obtained:

$$H=\frac{2kPx^{(2-n)}}{3}\left[\frac{1}{a^2+x^2}-\frac{1}{b^2+x^2}\right] \qquad (13)$$

$$M=\frac{2kPx^{(1-n)}}{3}\left[\frac{(b-a)x}{a^2+x^2}+\tan^{-1}\frac{a}{x}-\tan^{-1}\frac{b}{x}\right] \qquad (14)$$

For $a=0$, equation (14) becomes

$$M=\frac{2kPx^{(1-n)}}{3}\left[\frac{b}{x}-\tan^{-1}\frac{b}{x}\right] \qquad (15)$$

It will be found that for a problem such as that of figure 4 equations 13, 14, and 15 will give results only slightly greater than those obtained from the exact equations 2 and 4 to 10, particularly for relatively small values of a and x. For example, for the values $b=30$ feet, $a=0.5$ foot, $x=1.18$ feet, the moment computed by equation 14 was found to be less than 1 percent greater than that computed by equations 4 to 10. Moments and forces determined by the simpler

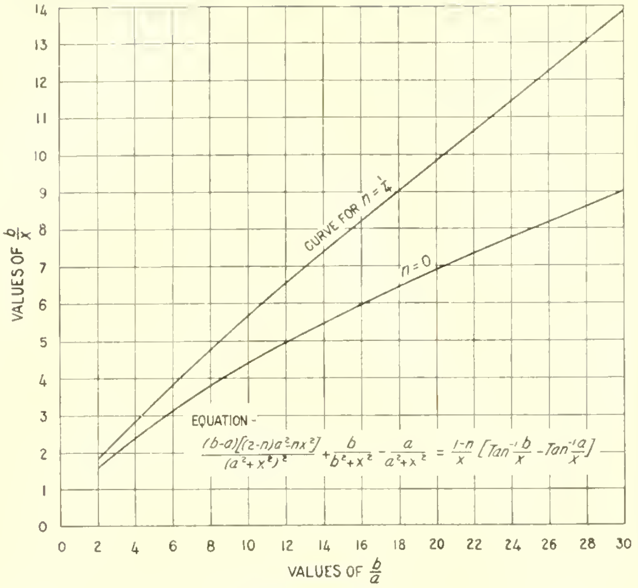


FIGURE 5.—CURVES SHOWING POSITION OF LOAD FOR MAXIMUM MOMENT, $a>0$.

expressions will be on the safe side and it is believed that their use is justified.

Up to this point in the discussion the distance x has been treated only as a constant. However, for live loads moving toward or away from an abutment there is a value of x for which the moment is a maximum. To derive an equation for this value of x, equation 14 may be differentiated with respect to x and the result equated to zero. Performing these operations the following expression is obtained:

$$\frac{(b-a)[(2-n)a^2-nx^2]}{(a^2+x^2)^2}+\frac{b}{b^2+x^2}-\frac{a}{a^2+x^2}=\frac{1-n}{x}\left(\tan^{-1}\frac{b}{x}-\tan^{-1}\frac{a}{x}\right)\text{----------}(16)$$

Substituting known values for b and a in the equation enables solving for x. Placing the load at this distance from the wall will then produce the maximum moment. If the value of n is taken as ¼, equation 16 becomes

$$\frac{(b-a)(7a^2-x^2)}{4(a^2+x^2)^2}+\frac{b}{b^2+x^2}-\frac{a}{a^2+x^2}=\frac{3}{4x}\left(\tan^{-1}\frac{b}{x}-\tan^{-1}\frac{a}{x}\right)\text{---------}(17)$$

While solving for x appears to be involved, the equation may be handled practically by plotting two curves, one for each side of the equation. Extending the two curves until they intersect will give the solution, the point of intersection being the value of x for which the equality is true. The solution of equation 17 is given in figure 5 where the ratio $\frac{b}{x}$ is plotted against the ratio $\frac{b}{a}$. When b and a are known, $\frac{b}{x}$ may be found from figure 5 and x may be determined from the ratio.

When the top of the fill coincides with the top of the wall the value of a is zero. For this particular case equation 16 for the maximum moment is of no use. It will be seen that for this case the moment approaches infinity as x approaches zero. This is due to the fact that in theory, as the point of application of the force is approached, the finite force is acting upon an infinitely small area. Practically, however, the load P is not concentrated at a point but is distributed over an area. If the area of distribution is assumed to be a 15-inch circle, it is obvious that the value of x cannot be less than the radius of the circle. Accordingly, the minimum value of x was taken as 7½ inches or 0.625 foot.

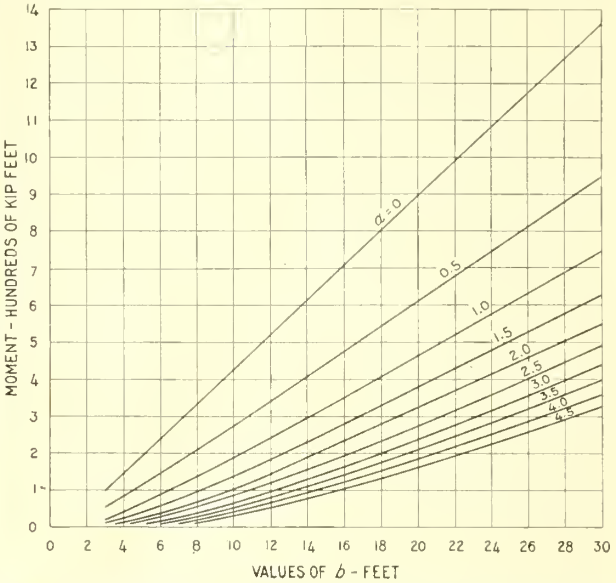


FIGURE 6.—MAXIMUM OVERTURNING MOMENTS AT VARIOUS DISTANCES FROM TOP OF FILL FROM THE REAR WHEELS OF TWO H-15 TRUCKS, $k=1.3$, $n=\frac{1}{4}$.

METHOD OF IMAGES MAY BE USED

Using four concentrated loads of 12,000 pounds each, a value of k equal to 1.3, n equal to ¼, and values of x for the maximum moment, the curves of figure 6 were plotted. It will be noted that the distance a has a considerable effect upon the moment, and that for large values of a the moment is greatly reduced.

In dealing with the standard H-loading it may be necessary in some cases to consider the smaller wheel concentrations 14 feet distant from the large wheels. It is unlikely, however, that two trucks headed in the same direction will enter a bridge at the same instant. Assuming that the four wheels shown in figure 4 are rear wheels of trucks, the front wheels of one truck will be on the fill and the front wheels of the other truck will be on the superstructure. Under such conditions the effect of the two front wheels on the fill will usually be very small in comparison with the effect of the four rear wheels which are close to the wall. However, if it is desired to include the effect of all wheels, the rear wheels may be placed at the point for maximum moment and with the trucks in this position a separate computation made for each set of loads. This position of the trucks should give moments sufficiently close to a maximum for design purposes.

When the total moment and the total horizontal force have been computed the height of the center of pressure may be determined by dividing the moment by the force. The ratio of this height to the height of wall, $(b-a)$, was found to vary from approximately 0.53 to 0.96 as the height of wall varies from 2 to 30 feet. Further, this ratio increases when the height of wall increases or when the distance a decreases. It is noted that the location of the center of pressure is not affected by either of the constants k or n.

Some designers may prefer to assume a soil of perfect elasticity and to use the equation based upon the theory of elasticity rather than the constants determined by experiment. In this case the method of images may be applied as explained by Dr. Raymond D. Mindlin.[4] The constant n will drop out or become equal to zero and the constant k which appears in the above equations will become $\frac{3}{\pi}$. By this method the intensity of normal pressure at any point on the wall is simply double that given by the equation for the stress in an elastic, isotropic solid (see p. 167). The force and moment equations 2 to 15, and equation 16 for the maximum, may be used by making this change in the constants. The values obtained for the force and moment will be slightly less than for $k=1.3$, $n=1/4$, when the load is placed at the point for maximum moment.

Assuming a bridge abutment backfilled with gravel, the moment and horizontal force will be computed by the approximate equations. The conditions assumed will be a live load P of 24 kips, $b=20$ feet, $a=1$ foot, $k=1.3$, and $n=1/4$.

The moment, using equation 14, is

$$M=\frac{2}{3}kPx^{1-n}\left[\frac{(b-a)x}{a^2+x^2}-\left(\tan^{-1}\frac{b}{x}-\tan^{-1}\frac{a}{x}\right)\right]$$

From figure 5 for $\frac{b}{a}=20$, $\frac{b}{x}=9.82$.

Then $x=\frac{20}{9.82}=2.03$ feet, the position of the load for maximum moment when $n=1/4$,

$$(2.03)^{1-n}=(2.03)^{3/4}=1.70$$

and

$$\frac{2}{3}kPx^{1-n}=\frac{2}{3}(1.3)24(1.70)=35.4$$

$$M=35.4\left[\frac{19(2.03)}{5.12}-(\tan^{-1}9.82-\tan^{-1}0.492)\right].$$

$$\text{Tan}^{-1}\,9.82-\tan^{-1}\,0.492=(84°12')-(26°12')=58.0°=0.01745(58.0)=1.01 \text{ radians.}$$

$$M=35.4(7.52-1.01)=230 \text{ foot-kips.}$$

This moment may be determined from figure 6. The value read from the chart should be divided by 2 since a load of 48 kips was used in computing values for the curves.

By equation 13

$$H=\frac{2}{3}kPx^{1-n}x\left[\frac{1}{a^2+x^2}-\frac{1}{b^2+x^2}\right],$$

the horizontal force

$$H=35.4(2.03)\left[\frac{1}{5.12}-\frac{1}{404.1}\right]=71.8(0.1952-0.0025)=13.8 \text{ kips.}$$

If an elastic, isotropic backfill material is assumed and the live load effect on the wall is taken into account by using the method of images the only change in the problem will be in the assumed values of the constants. The new values will be $k=\frac{3}{\pi}$ and $n=0$. Again taking $P=24$ kips, $b=20$ ft., and $a=1$ foot, from figure 5 for $\frac{b}{a}=20$, $\frac{b}{x}=6.90$.

Then $x=\frac{20}{6.90}=2.80$ feet, the position of the load for a maximum moment when $n=0$.

$$\frac{2}{3}kPx^{1-n}=\frac{2}{3}\cdot\frac{3}{\pi}(24)2.80=42.8$$

$$M=42.8\left[\frac{19(2.80)}{8.84}-(\tan^{-1}6.90-\tan^{-1}0.357)\right]$$

$$\text{Tan}^{-1}\,6.90-\tan^{-1}\,0.357=(81°46')-(19°38')=62.13°=0.01745(62.13)=1.08 \text{ radians}$$

$$M=42.8(6.02-1.08)=211 \text{ foot-kips}$$

and

$$H=42.8(2.80)\left[\frac{1}{8.84}-\frac{1}{407.8}\right]=119.5\ (0.1130-0.0024)=13.2 \text{ kips.}$$

The above computations were made with an ordinary 10-inch slide rule using C, D, L, and T scales.

CONCLUSIONS

In the preparation of figure 6 the value of k was taken as 1.3 as this is the value suggested as safe for gravel material similar to that used in the experiments.[3] In the absence of further experimental data this value might be used for the average gravel backfill.

Regarding the experimental constant n, the value of 1/4 was found to agree most closely with the experimental data reported by Spangler. Since this constant is written as an exponent of x, its value affects the position of the load for maximum moment. With reference to the curves of figure 5 it will be noted that for a maximum moment the load must be placed farther from the wall for n equal to zero than for n equal to ¼.

The reader may question when to use the method involving the empirical constants and when to use the method of images. It is the writer's opinion that when dealing with a backfill whose soil properties are unknown it would be more logical to assume elasticity and use the method of images than to make a guess at a value for k which had not been verified by experiment.

In this study an attempt has been made to shorten the gap between theory and practice by reducing the amount of mathematical work involved in applying the theory. Since the equations derived are in conformity with measured pressure intensities it is believed that by the use of these equations the effect of concentrated loads can be more accurately predicted than by rule of thumb methods.

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[4] Dr. Raymond D. Mindlin. Proceedings of the International Conference on Soil Mechancis and Foundation Engineering. vol. III. 1936, p. 155.

[3] Horizontal Pressures on Retaining Walls Due to Concentrated Surface Loads, by M. G. Spangler. Iowa Engineering Experiment Station Bulletin 140. 1938.

(*Continued from page 166*)

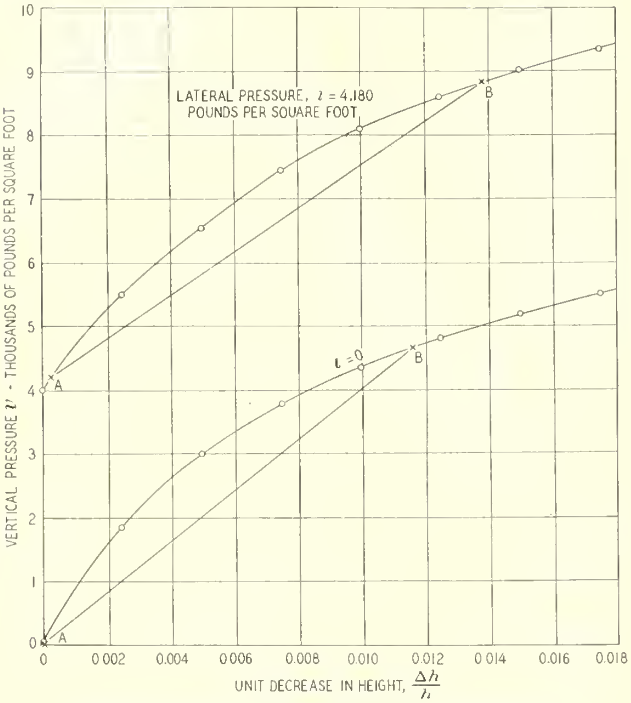

Figure 8.—Triaxial Test Data for Fill Soil.

lower curve, is $\frac{2,984-0}{0.0235+0.0001}=126,000$ pounds per square foot $=C$. The average modulus C is then taken as $\frac{109,000+126,000}{2}=118,000$ pounds per square foot.

For $\frac{z}{B}=1.97$, $\mu=\frac{1}{2}$ and $\frac{b}{B}=0.453$, it is seen from figure 5 that $F=0.73$. Then, from equation 18, S_L (undersoil) $=\frac{pB}{C}F=\frac{4,662\times 50.8}{118,000}\times 0.73=1.46$ feet.

S_L within the fill itself is now computed and equation 11, taking $K'=0$, is used for this purpose. Assume that the lateral pressure is zero from 0′ to 0, figure 6. This assumption is on the side of safety.

The vertical pressure at 0′ is zero whereas at 0 it is equal to p or 4,662 pounds per square foot as already shown. To be consistent with the procedure followed in computing S_L in the undersoil, it is assumed that the point B on each of the two curves, figure 8, corresponds to $v-l=v-0=4,662$ pounds, the maximum $v-l$ in the fill. Figure 8 shows the v versus $\frac{\Delta h}{h}$ characteristics of the compacted fill soil. For the upper curve, figure 8, C is computed as 343,000 pounds per square foot and for the lower curve its value 398,000 pounds per square foot, the average value being 370,000 pounds per square foot. Substituting in equation 11, $S_L(\text{fill})=\frac{3w}{8C}H^2=\frac{3\times 126}{8\times 370,000}\times 37\times 37=0.18$ foot.

The total settlement due to soil deformation is then $S_L(\text{fill})+S_L(\text{undersoil})=0.18+1.46=1.64$ feet.

Due to deformation of soil at constant volume, the elevation of the roadway is therefore diminished by 1.64 feet. A considerable part of this settlement would most likely occur during construction of the embankment. To obtain the total settlement, to the settlement S_L must be added the settlement S_C caused by volume changes in the fill and undersoil.

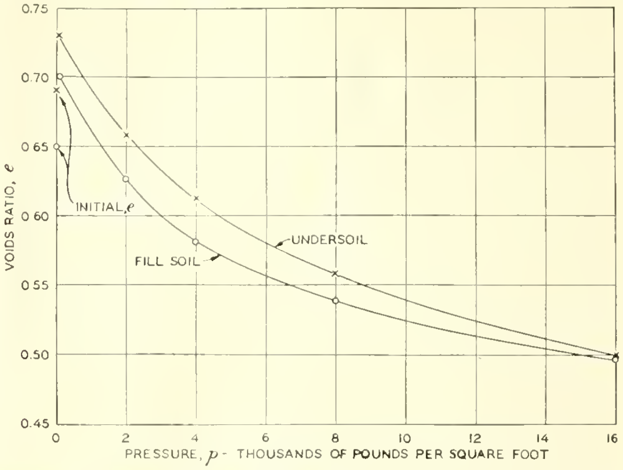

Figure 9.—Consolidation Test Data for Fill and Foundation Soils.

COMPUTATION OF S_C ILLUSTRATED

In figure 9, the voids ratio, e, versus pressure curves are shown for the two soils, fill soil and undersoil. The initial voids ratios of the undisturbed core samples at their natural moisture contents are shown on the e axis in this figure. These are the e values prior to making the consolidation test. For the foundation soil, the initial e is 0.69 and for the fill soil, the initial e is 0.65.

For computing the settlement S_C, the following expression is used,

$$S_C=\frac{e_i-e_f}{1+e_i}\times D \qquad (19)$$

where e_i is the average voids ratio within the thickness D of the soil mass prior to its consolidation and e_f is the average voids ratio within the same soil mass when consolidation is complete. In this problem, D is the height of fill, 37 feet, when computing S_C within the fill and the thickness of the compressible layer of undersoil, 100 feet, when computing S_C in the undersoil.

For the fill, e_i is taken as the initial value, 0.65, the voids ratio of the soil as compacted in the fill. In computing e_f for the fill, it is assumed that the compacted soil expands at the road surface from $e=0.65$ to $e=0.70$, as shown in figure 9, this expansion being due to wetting. At the base of the fill, the final value of e corresponds to the fill load of 4,662 pounds per square foot and from figure 9, this value of e is 0.57. Then e_f, the average final value of e is $\frac{0.70+0.57}{2}=0.635$. Then S_C (fill) $=\frac{0.65-0.635}{1+0.65}\times 37=0.34$ foot.

For the undersoil, it is assumed that the clay soil is consolidated under its own weight prior to fill construction. The initial voids ratio of an undisturbed core sample taken at the subgrade surface is seen to be 0.69 from figure 9. At a depth of 100 feet, the vertical pressure due to overburden is 100 × weight per cubic foot of undersoil $=100\times 128=12,800$ pounds per square foot, corresponding to an e of 0.52. Therefore, $e_i=\frac{0.69+0.52}{2}=0.605$, the average e prior to consolidation under the fill load. After complete consolidation under the fill load, e at the point 0, figure 6, corresponds to a pressure of 4,662 pounds per square foot. This value of e, figure 9, is 0.60.

At the bottom of the compressible undersoil layer, point B, figure 6, the final value of e corresponds to a pressure of 12,800 pounds per square foot due to overburden plus the pressure transmitted by the weight of the fill. The latter pressure is the product, pf, where $p=4{,}662$ pounds per square foot and f is obtained from figure 4. To find f, remember that $\frac{b}{B}=0.453$ and $\frac{z}{B}=1.97$ in this problem. Thus $pf=4{,}662\times0.54=2{,}517$ pounds per square foot. Then, the total vertical pressure at the lower boundary of the compressible supporting layer of soil is $12{,}800+2{,}517=15{,}317$ pounds per square foot corresponding to an e of 0.50. Then $e_f=\frac{0.60+0.50}{2}=0.55$, the average e from top to bottom of the supporting soil subsequent to complete consolidation under the weight of the fill. Using equation 19, S_C (undersoil)$=\frac{0.605-0.550}{1+0.605}\times100=3.43$ feet.

The total S_C is then S_C (fill)$+S_C$ (undersoil)$=0.34+3.43=3.77$ feet and total S_L+ total $S_C=1.64+3.77=5.41$ feet.

SETTLEMENT RECORDS OF EMBANKMENTS NEEDED

The purpose of presenting formulas relating to the bearing capacities and displacements of soils is to enable the engineer to make a reasonably accurate approximation. It would be a serious mistake to rely completely on any mathematical formula derived from theory and assumptions without reference to observations in the field and to practical experience. Where the problem deals with large earth masses, mathematical expressions can at the best only indicate the general trend of physical occurrences. After many years, the proposed formulas may be modified and accepted or they may be completely rejected depending upon the extent to which, on trial, they have proved to be serviceable. The details in the development of the formulas serve to establish and clarify certain fundamental principles and conceptions which are often of more value than the formulas themselves.

The theoretical expressions for computing settlement caused by lateral displacement, such as equations 11, 16, and 17 would be expected to be more precise for relatively small deformations. As deformations increase in magnitude one or more points within the soil mass may become stressed to a condition characteristic of failure while the soil surrounding these points is stressed below this limit. As soon as a condition of failure is reached anywhere in the soil mass, even though it be reached at only one point, the whole system of stresses throughout the soil in the loaded region changes and the stress-deformation relations expressed by equations 11, 16, and 17 cannot be considered as a true picture of conditions. However, the true limitations of these theoretical expressions which are based on an assumption of a strictly linear relationship between stress and deformation are not known. With certain soils, the relationship may never be linear for stresses of any magnitude whatsoever.

The essential requirement is a correlation of field observations with theoretical and laboratory studies. Computed settlements such as those presented in the example must be checked against observed settlements. Theory may serve as a rough guide or crude yardstick in promoting more thorough and intelligent field studies. This is about the best that can be expected of its use.

DISPOSITION OF STATE MOTOR-FUEL TAX RECEIPTS, 1939

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration	For other administrative purposes[3]	For State highway purposes						For local roads and streets[6]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes					
						Construction, maintenance, and administration[4]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads[4]	For work on city streets[7]	Service of local highway obligations	Total		To general funds[8]		For relief of unemployment or destitution	For education	For other purposes[9]	Total
								State highway bonds and notes	Reimbursement obligations[5]	Total							Inspection fees, dealers licenses, etc.	Motor-fuel tax				
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	14,476	−79	14,397	34	61	5,103		2,070		2,070	7,173	7,129			7,129							
Arizona	4,446	78	4,524	60		2,989	116				3,105	*1,357			1,357						2	2
Arkansas	10,742		10,742	344	3	2,959	21	4,288	1,818	6,106	9,086	1,123		186	1,309							
California	49,811	−916	48,895	184		28,664					28,664	*13,226	3,914		17,140				2,907			2,907
Colorado	7,751	32	7,783	97	24	3,761	270	1,555		1,555	5,586	2,076			2,076							
Connecticut	9,961	39	10,000	27		8,366			1,607	1,607	9,973											
Delaware	2,146		2,146	10		[4] 1,395	201	154	372	526	2,122	(4)	1		1						13	13
Florida	24,767	941	25,708	32	91	12,795			8,958	8,958	21,753						319	1,738		1,737	38	3,832
Georgia	21,088	−8	21,080	350		9,928			19	19	9,947	3,385			3,385		31	3,634		3,733		7,398
Idaho	4,464	−134	4,330	19	3	3,090	17	1		1	3,108	1,200			1,200							
Illinois	39,400	20	39,420	182	237	12,025					12,025	6,839	9,631	3,791	20,261		162		4,385	2,168		6,715
Indiana	24,318	2	24,320	106	116	12,922	643				13,565	7,835	1,453		9,288		376	869				1,245
Iowa	14,033	−60	13,973	68	60	3,798			3,352	3,352	7,150	*6,695			6,695							
Kansas	10,179	−32	10,147	338	108	6,214	123	228	679	907	7,244	2,457			2,457							
Kentucky	13,836	−6	13,830	39		11,863	244				12,107	1,684			1,684							
Louisiana	17,463	1,495	18,958	37	81			10,696		10,696	10,696							2,723	2,760	1,331	1,330	8,144
Maine	5,718		5,718	16		3,279	211	1,610		1,610	5,100	602			602							
Maryland	10,638		10,638	36		4,940		1,442		1,442	6,382	915	2,692	613	4,220							
Massachusetts	20,512	−274	20,238	50		2,556	376	[10] 3,344	5	3,349	6,281	9,483		[10] 1,789	11,272	1,239			[10] 1,389		7	1,396
Michigan	29,794	150	29,944	260		18,502		4,627		4,627	23,129	*6,550			6,550		5					5
Minnesota	18,813	−90	18,723	27	112	12,070	303				12,373	*6,186			6,186		25					25
Mississippi	11,309	1	11,310	103	10	2,574	175	3,241		3,241	5,990	*4,713			4,713	494						
Missouri	12,234		12,234	58	52	6,017	255	5,725	48	5,773	12,045						79					79
Montana	4,807	−103	4,704	23	7	3,702		972		972	4,674											
Nebraska	11,585	3	11,588	121	33	5,903					5,903	*3,430	342		3,772				1,759			1,759
Nevada	1,359		1,359	4	23	1,324			8	8	1,332											
New Hampshire	3,508	−1	3,507	(11)		1,750	109	879		879	2,738	640		[12] 129	769							
New Jersey	23,068	203	23,271	196		7,845	296	2,285		2,285	10,426	4,144		889	5,033	257		[13] 3,000	3,290	580	489	7,359
New Mexico	4,320	50	4,370	95	3	2,401		1,871		1,871	4,272											
New York[14]	69,757	265	70,022	107		7,959	591	4,695		4,695	13,245	*8,484	1,743		10,227	460		45,983				45,983
North Carolina	25,556	4	25,560	(15)	102	[4] 17,512		6,635	283	6,918	24,430	(4)					915				113	1,028
North Dakota	2,736	−23	2,713	21	66	1,840					1,840	760			760		22	4				26
Ohio	[16] 50,466	−1,570	48,896	69		20,082					20,082	9,019	6,510		15,529			11,463		1,753		13,216
Oklahoma	14,495	−283	14,212	419	137	9,913					9,913	*3,220	373		3,593		150					150
Oregon	10,582	−268	10,314	43		6,209	246	2,003	9	2,012	8,467	*1,708	10		1,718	86						
Pennsylvania	59,590		59,590	211		25,947	2,520	5,071		5,071	33,538	*10,680			10,680	95		14,861			205	15,066
Rhode Island[17]	4,031		4,031	15		1,148	127	159		159	1,434	35			35	40		2,507				2,507
South Carolina	12,516		12,516	44	30	7,519		1,468	1,056	2,524	10,043	*2,065	158		2,223		176					176
South Dakota	4,012	40	4,052	54	54	3,410	15	9		9	3,434	491			491	6	13					13
Tennessee	19,727	−736	18,991	306	41	2,486		6,432	1,441	7,873	10,359	5,389			5,389	22	1,034				1,840	2,874
Texas	45,208	−63	45,145	697		20,058	537		11,117	11,117	31,712	1,640			1,640					11,096		11,096
Utah	3,688	51	3,739	9	9	3,584	137				3,721											
Vermont	2,679		2,679	3		1,399		294		294	1,693	962			962	6					15	15
Virginia	17,829		17,829	186	32	[4] 16,649	309		263	263	17,221	[4] 298	79		377						13	13
Washington	16,006		16,006	50		6,719		[10] 118	52	170	6,889	*5,900	1,806	[10] 362	8,068				[10] 999			999
West Virginia	10,423	−14	10,409	15		[4] 5,584		4,810		4,810	10,394	(4)										
Wisconsin	20,438	−3,746	16,692	75	167	5,851			2,360	2,360	8,211	5,052	712		5,764	141		2,334				2,334
Wyoming	2,580		2,580	11		1,745	51	112		112	1,908	661			661							
District of Columbia	2,791	5	2,796	(18)									2,789		2,789		7					7
Total	821,656	−5,027	816,629	5,251	1,662	364,349	7,893	76,794	33,447	110.241	482,483	148,033	32,213	7,759	188,005	2,846	3,314	89,116	17,489	22,398	4,065	136,382

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See following tables.

[3] Where reported separately from collection expenses, funds allotted for motor-fuel inspection, administration of motor vehicle department, and regulation of motor vehicles are shown in this column.

[4] The following amounts for construction and maintenance of county roads under State control are included in allotments for State highway purposes: Delaware, $279,000; North Carolina, $7,367,000; Virginia, $6,554,000; West Virginia, $2,082,000. In Virginia the 3 counties whose roads are not under State control received $298,000.

[5] Reimbursement to counties and local units of government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[8] To State general funds, except as follows: Louisiana, 1 cent of tax to parishes; Wisconsin, payment to towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, State parks, CCC ditching, etc.; Florida, aviation; Louisiana and Massachusetts, harbor improvement; New Jersey, debt service on institutional construction bonds, $268,000, department of commerce and navigation, $166,000, other departments, $55,000; North Carolina, State probation and parole commissions; Pennsylvania, aviation, $171,000, cooperative work other departments, $34,000; Tennessee, debt service on nonhighway bonds; Vermont, debt service on nonhighway portion of flood relief bonds; Virginia, aviation.

[10] Includes debt service charges on emergency relief bond issues, prorated in proportion to use of proceeds for State highways, local roads and streets, and nonhighway purposes.

[11] Paid out of motor-vehicle revenue, $6,000. See following table.

[12] Debt service on highway relief bonds, a State obligation incurred for improvement of local roads.

[13] Originally appropriated for relief but later transferred by legislative action to State general fund.

[14] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle revenues to the general fund and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

[15] Included in cost of collecting motor-vehicle revenue. See following table.

[16] Tax of $641,000 on nonmotor-vehicle fuels not included.

[17] Expenditures for highway purposes have been credited against payments of motor-fuel tax and motor vehicle revenues to the State general fund and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

[18] Paid out of general revenue. Amount not reported.

DISPOSITION OF STATE MOTOR-VEHICLE RECEIPTS, 1939

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed [2]	Expenses of collection and administration [3]	For other administrative purposes [4]	For State highway purposes						For local roads and streets [7]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes				
								Service of State highway obligations													
						Construction, maintenance, and administration [5]	State highway police	State highway bonds and notes	Reimbursement obligations [6]	Total	Total for State highway purposes	For work on county and local roads [5]	For work on city streets[8]	Service of local highway obligations	Total		To general funds [9]	For relief of unemployment or destitution	For education	For other purposes [10]	Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	4,947	−10	4,937	434		1,940	632	978		978	3,550		180		180		773				773
Arizona	1,134	−3	1,131	255		843	33				876										
Arkansas	3,140	673	3,813	105		1,096	174	1,640	759	2,399	3,669			39	39						
California	24,646	−172	24,474	3,090	307	3,760	2,889	3,785		3,785	10,434	*3,781			3,781		6,862				6,862
Colorado	2,879	1	2,880	730		750	53	303		303	1,106	1,044			1,044						
Connecticut	6,738	85	6,823	857		2,289	286		440	440	3,015	2,951			2,951						
Delaware	1,313	−512	801	106		[5] 453	66	50	121	171	690	(5)	1		1					4	4
Florida	7,096		7,096	524	217		191				191								6,164		6,164
Georgia	2,620	−153	2,467	300		880	526				1,406	694			694		67				67
Idaho	1,399	−33	1,366	89		114	134				248	*1,029			1,029						
Illinois	24,352	857	25,209	1,336	369	11,294	1,500	9,510		9,510	22,304	1,047			1,047		153				153
Indiana	10,001	242	10,243	1,053		5,006	249				5,255	3,035	563		3,598		337				337
Iowa	12,624	1	12,625	1,442		5,541			4,890	4,890	10,431	752			752						
Kansas	4,363	−180	4,183	602		2,293	46	84	251	335	2,674	907			907						
Kentucky	4,960	1	4,961	485	26	2,021	56				2,077	886			886		1,487				1,487
Louisiana	5,309	−201	5,108	163	50	4,022	435	438		438	4,895										
Maine	3,788	−169	3,619	160		1,990	128	976		976	3,094	365			365						
Maryland	5,538	1	5,539	341	54	2,106	399	1,143		1,143	3,648		852		852		244	400			644
Massachusetts	7,028	251	7,279	1,587	70	712	104	[11] 931	2	933	1,749	2,641		[11] 498	3,139	345		[11] 387		2	389
Michigan	22,119	−182	21,937	957	408		494				494	*19,932			19,932		146				146
Minnesota	9,884	−116	9,768	573		4,573	115	2,338	2,031	4,369	9,057						138				138
Mississippi	2,567		2,567	175		230					230	*2,162			2,162						
Missouri	10,226	−9	10,217	615		5,423	230	3,905	44	3,949	9,602										
Montana	1,634	−52	1,582	79			251				251	1,211	41		1,252						
Nebraska	2,683	−167	2,516	214	18	672	74				746	1,538			1,538						
Nevada	286		286	32		150	20	84		84	254										
New Hampshire	2,962	28	2,990	132	6	1,871	156	3		3	2,030	684		[12] 138	822						
New Jersey	21,312	182	21,494	2,049		7,598	287	2,214		2,214	10,099	4,014		861	4,875	248		3,187	562	474	4,223
New Mexico	1,909	−222	1,687	151	119	576	108				684	184	58		242		491				491
New York [13]	48,432	−38	48,394	2,509	149	10,276	764	6,062		6,062	17,102	*9,072	4,581		13,653	594	14,387				14,387
North Carolina	8,163	−213	7,950	492		[5] 4,976	517	1,885	80	1,965	7,458	(5)									
North Dakota	1,619	−18	1,601	128		747	77				824	649			649						
Ohio	28,625	1,556	30,181	2,780		7,375	825				8,200	*14,177	4,719		18,896					305	305
Oklahoma	6,046	−155	5,891	782	121	1,310	621				1,931	*2,402	655		3,057						
Oregon	3,469	−15	3,454	471		1,804	71	582	2	584	2,459	*496	3		499	25					
Pennsylvania	36,563	[14] −19,215	17,348	1,664		10,987	1,068	2,147		2,147	14,202	1,355			1,355	40				87	87
Rhode Island [15]	2,929	−4	2,925	215		774	86	107		107	967	24			24	27	1,692				1,692
South Carolina	1,915		1,915	170		1,024	369	183	144	327	1,720	3	22		25						
South Dakota	2,165	24	2,189	87		336	1	1		1	338	1,298			1,298	1	465				465
Tennessee	5,443	−262	5,181	850		3,305	221				3,526	77			77	14	714				714
Texas	21,194	64	21,258	1,434		7,138	191				7,329	*12,495			12,495						
Utah	1,176	36	1,212	132	14	9		208		208	217	560	240		800		49				49
Vermont	2,490		2,490	64		1,207	116	254		254	1,577	830			830	5				14	14
Virginia	6,847	78	6,925	334		6,191	190		162	162	6,543		48		48						
Washington	4,560	31	4,591	398		2,567	1,476				4,043	12			12	138					
West Virginia	5,974		5,974	206		[5] 2,792	31	2,945		2,945	5,768	(5)									
Wisconsin	12,954	−2,537	10,417	728	24	3,436			1,386	1,386	4,822	2,967	418		3,385	84	1,374				1,374
Wyoming	622	37	659	13		436	13	197		197	646										
District of Columbia	1,851		1,851	136	55								1,660		1,660						
Total	412,494	−20,490	392,004	32,229	2,007	134,893	16,273	42,953	10,312	53,265	204,431	95,274	14,041	1,536	110,851	1,521	29,379	3,974	6,726	886	40,965

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See preceding and following tables.

[3] Collection expenses in many States include service charges deducted by county and local collectors.

[4] Where reported separately from collection expenses, funds allotted for collection of motor-fuel tax, payments to auto theft fund, and miscellaneous expenses of motor-vehicle regulation are shown in this column.

[5] The following amounts for construction and maintenance of county roads under State control are included in allotments for State highway purposes: Delaware, $91,000; North Carolina, $2,093,000; West Virginia, $648,000.

[6] Reimbursement to counties and local units of government for amounts spent on roads now on State system.

[7] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[8] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[9] To State general funds except in the following States: Alabama, county and municipal general funds; California, general funds of counties and cities, $3,776,000; New Mexico, county general funds, $307,000; Wisconsin, towns, cities, and villages in lieu of personal property taxes formerly imposed on motor vehicles, $1,370,000. Allocations to county and local general funds may have been used in part for highways, but such amounts not reported.

[10] For the following purposes: Delaware, State parks, CCC ditching, etc.; Massachusetts, harbor improvement; New Jersey, debt service on institutional construction bonds, $259,000, Department of Commerce and Navigation, $162,000, other departments, $53,000; Ohio, care of indigent persons injured in motor-vehicle accidents; Pennsylvania, aviation, $73,000, cooperative work other departments, $14,000; Vermont, debt service on nonhighway portion of flood relief bonds.

[11] Includes debt service charges on emergency relief bond issues, prorated in proportion to use of proceeds for State highways, local roads and streets, and nonhighway purposes.

[12] Debt service on highway relief bonds, a State obligation incurred for improvement of local roads.

[13] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle revenues to the general fund and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

[14] Due to change in registration year from Jan. 1 to Apr. 1.

[15] Expenditures for highway purposes have been credited against payments of motor-fuel tax and motor-vehicle revenues to the State General Fund and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

DISPOSITION OF STATE MOTOR-CARRIER TAX RECEIPTS, 1939

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed [2]	Expenses of collection and administration	For State highway purposes						For local roads and streets [5]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes		
					Construction, maintenance, and administration [3]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads [3]	For work on city streets [6]	Service of local highway obligations	Total		To general funds	For education	Total
							State highway bonds and notes	Reimbursement obligations [4]	Total									
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	252	−59	193	44	149					149								
Arizona	187	−1	186	37	143	6				149								
Arkansas	3		3	2	1					1								
California	3,032	−8	3,024	421	78					78	5			5		2,520		2,520
Colorado	728	30	758	127	253	18	105		105	376	255			255				
Connecticut	274	−8	266	14	107			21	21	128	124			124				
Delaware	(7)																	
Florida	337	14	351	73								10	262	272			6	6
Georgia	86	−19	67	57	3					3						7		7
Idaho	60	−9	51	21		30				30								
Illinois	(7)																	
Indiana	1,222	86	1,308	106	655	33				688	397	73		470		44		44
Iowa	558		558	166							*392			392				
Kansas	1,313		1,313	321	636	12	23	70	93	741	251			251				
Kentucky	293		293	53	207	4				211	29			29				
Louisiana	93		93	20												73		73
Maine	18		18	18														
Maryland	(8)																	
Massachusetts	140		140	45												95		95
Michigan	465	37	502	210	292					292								
Minnesota	41		41	41														
Mississippi	116		116	34							*82			82				
Missouri	794	144	938	99	694	29		6	6	729	26	84		110				
Montana	61	−6	55	55														
Nebraska	28		28	28														
Nevada	223	2	225	12	189	24				213								
New Hampshire	3		3	3														
New Jersey	100	−6	94		94					94								
New Mexico	193	−73	120	48	72					72								
New York	(7)																	
North Carolina	365		365	12	[3] 253		96	4	100	353	(3)							
North Dakota	36		36	36														
Ohio	522	76	598	119	297					297	182			182				
Oklahoma	1,502	8	1,510	130	716					716	664			664				
Oregon	1,219	−17	1,202	191	592	55	191	1	192	839	*163	1		164	8			
Pennsylvania	8		8		8					8								
Rhode Island	11		11	7	1					1						3		3
South Carolina	256	1	257	68	167					167		22		22				
South Dakota	453	162	615	32	558	2	1		1	561					1	21		21
Tennessee	184	20	204	42	103					103	5			5	1	53		53
Texas	115		115	104	11					11								
Utah	(9)	1	1	1														
Vermont	(7)																	
Virginia	330		330	62	252	8		6	6	266		2		2				
Washington	195		195	195														
West Virginia	103		103		43		60		60	103								
Wisconsin	1,675	−152	1,523	251												1,272		1,272
Wyoming	248	5	253	24	223	6				229								
District of Columbia	213		213													213		213
Total	18,055	228	18,283	3,329	6,797	227	476	108	584	7,608	2,575	192	262	3,029	10	4,301	6	4,307

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[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See preceding tables

[3] Approximately $106,000 for use on county roads under state control in North Carolina included in allotments for State highway purposes.

[4] Reimbursement to counties and local units of government for amounts spent on roads now on State system.

[5] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[6] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[7] No special taxes on motor carriers reported.

[8] Ton-mile and passenger-mile taxes paid by motor carriers in lieu of registration fees included in motor-vehicle receipts, preceding table.

[9] Motor-carrier taxes no longer imposed in Utah.

DISPOSITION OF RECEIPTS FROM STATE IMPOSTS ON HIGHWAY USERS, 1939

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year [1]	Adjustments due to undistributed funds, etc. [2]	Net total funds distributed	For State highway purposes							For local roads and streets [6]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes					
							Service of State highway obligations									To general funds [8]					
				Expenses of collection and administration [3]	Construction, maintenance, and administration [4]	State highway police	State highway bonds and notes	Reimbursement obligations [5]	Total	Total for State highway purposes	For work on county and local roads [4]	For work on city streets [7]	Service of local highway obligations	Total		Motor-fuel inspection fees, dealers licenses, etc.	All other highway user imposts	For relief of unemployment or destitution	For education	For other purposes [9]	Total
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	19,675	−148	19,527	573	7,192	632	3,048		3,048	10,872	7,129	180		7,309			773				773
Arizona	5,767	74	5,841	352	3,975	155				4,130	*1,357			1,357						2	2
Arkansas	13,885	673	14,558	454	4,056	195	5,928	2,577	8,505	12,756	1,123		225	1,348							
California	77,489	−1,096	76,393	4,002	32,502	2,889	3,785		3,785	39,176	*17,012	3,914		20,926			9,382	2,907			12,289
Colorado	11,358	63	11,421	978	4,764	341	1,963		1,963	7,068	3,375			3,375							
Connecticut	16,973	116	17,089	898	10,762	286		2,068	2,068	13,116	3,075			3,075							
Delaware	3,459	−512	2,947	116	[4] 1,848	267	204	193	697	2,812	([4])	2		2						17	17
Florida	32,200	955	33,155	937	12,795	191		8,958	8,958	21,944		10	262	272		319	1,738		7,907	38	10,002
Georgia	23,794	−180	23,614	707	10,811	526		19	19	11,356	4,079			4,079		31	3,708		3,733		7,472
Idaho	5,923	−176	5,747	132	3,204	181	1		1	3,386	*2,229			2,229							
Illinois	63,752	877	64,629	2,124	23,319	1,500	9,510		9,510	34,329	7,886	9,631	3,791	21,308		162	153	4,385	2,168		6,868
Indiana	35,541	330	35,871	1,381	18,583	925				19,508	11,267	2,089		13,356		376	1,250				1,626
Iowa	27,215	−59	27,156	1,736	9,339			8,242	8,242	17,581	*7,839			7,839							
Kansas	15,855	−212	15,643	1,369	9,143	181	335	1,000	1,335	10,659	3,615			3,615							
Kentucky	19,089	−5	19,084	603	14,091	304				14,395	2,599			2,599			1,487				1,487
Louisiana	22,865	1,294	24,159	351	4,022	435	11,134		11,134	15,591							2,796	2,760	1,331	1,330	8,217
Maine	9,524	−169	9,355	194	5,269	339	2,586		2,586	8,194	967			967							
Maryland	16,176	1	16,177	431	7,046	399	2,585		2,585	10,030	915	3,544	613	5,072			244	400			644
Massachusetts	27,680	−23	27,657	1,752	3,268	480	[10] 4,275	7	4,282	8,030	12,124		[10] 2,287	14,411	1,584		95	[10] 1,776		9	1,880
Michigan	52,378	5	52,383	1,835	18,794	494	4,627		4,627	23,915	*26,482			26,482		5	146				151
Minnesota	28,738	−206	28,532	753	16,643	418	2,338	2,031	4,369	21,430	*6,186			6,186		25	138				163
Mississippi	13,992	1	13,993	322	2,804	175	3,241		3,241	6,220	*6,957			6,957	494						
Missouri	23,254	135	23,389	824	12,134	514	9,630	98	9,728	22,376	26	84		110		79					79
Montana	6,502	−161	6,341	164	3,702	251	972		972	4,925	1,211	41		1,252							
Nebraska	14,296	−164	14,132	414	6,575	74				6,649	*4,968	342		5,310				1,759			1,759
Nevada	1,868	2	1,870	71	1,663	44	84	8	92	1,799											
New Hampshire	6,473	27	6,500	141	3,621	265	882		882	4,768	1,321		[11] 267	1,591							
New Jersey	44,480	379	44,859	2,245	15,537	583	4,499		4,499	20,619	8,158		1,750	9,908	505		[12] 3,000	6,477	1,142	963	11,582
New Mexico	6,422	−245	6,177	416	3,049	108	1,871		1,871	5,028	184	58		242			491				491
New York [13]	118,189	227	118,416	2,765	18,235	1,355	10,757		10,757	30,347	*17,556	6,324		23,880	1,054		60,370				60,370
North Carolina	34,084	−209	33,875	606	[4] 22,741	517	8,616	367	8,983	32,241	([4])					915				113	1,028
North Dakota	4,391	−41	4,350	251	2,587	77				2,664	1,409			1,409		22	4				26
Ohio	79,613	62	79,675	2,968	27,754	825				28,579	*23,378	11,229		34,607			11,463		1,753	305	13,521
Oklahoma	22,043	−430	21,613	1,589	11,939	621				12,560	*6,286	1,028		7,314		150					150
Oregon	15,270	−300	14,970	705	8,605	372	2,776	12	2,788	11,765	*2,367	14		2,381	119						
Pennsylvania	96,161	[14] −19,215	76,946	1,875	36,942	3,588	7,218		7,218	47,748	*12,035			12,035	135		14,861			292	15,153
Rhode Island [15]	6,971	−4	6,967	237	1,923	213	266		266	2,402	59			59	67		4,202				4,202
South Carolina	14,687	1	14,688	312	8,710	369	1,651	1,200	2,851	11,930	*2,068	202		2,270		176					176
South Dakota	6,630	226	6,856	227	4,304	18	11		11	4,333	1,789			1,789	8	13	486				499
Tennessee	25,354	−978	24,376	1,239	5,894	221	6,432	1,441	7,873	13,988	5,471			5,471	37	1,034	767			1,840	3,641
Texas	66,517	1	66,518	2,235	27,207	728		11,117	11,117	39,052	*14,135			14,135					11,096		11,096
Utah	4,864	88	4,952	165	3,593	137	208		208	3,938	560	240		800			49				49
Vermont	5,169		5,169	67	2,606	116	548		548	3,270	1,792			1,792	11					29	29
Virginia	25,006	78	25,084	614	[4] 23,092	507		431	431	24,030	[4] 298	129		427						13	13
Washington	20,761	31	20,792	643	9,286	1,476	[10] 118	52	170	10,932	*5,912	1,806	[10] 362	8,080	138			[10] 999			999
West Virginia	16,500	−14	16,486	221	[4] 8,419	31	7,815		7,815	16,265	([4])										
Wisconsin	35,067	−6,435	28,632	1,245	9,287			3,746	3,746	13,033	8,019	1,130		9,149	225		4,980				4,980
Wyoming	3,450	42	3,492	48	2,404	70	309		309	2,783	661			661							
District of Columbia	4,855	5	4,860	191								4,449		4,449		7	213				220
Total	1,252,205	−25,289	1,226,916	44,478	506,039	24,393	120,223	43,867	164,090	694,522	245,882	46,446	9,557	301,885	4,377	3,314	122,796	21,463	29,130	4,951	181,654

[1] Includes receipts from motor-fuel taxes, motor-vehicle fees and fines, and special imposts on motor vehicles operated for hire (motor-carrier taxes). See preceding tables, which give distribution of receipts separately.

[2] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[3] Includes expenses of collection and administration of motor-fuel tax, motor-vehicle fees, and motor-carrier taxes, and miscellaneous expenses of motor-vehicle regulation.

[4] The following amounts for construction and maintenance of county roads under State control are included in allotments for State highway purposes: Delaware, $370,000; North Carolina, $9,566,000; Virginia, $6,554,000; West Virginia, $2,730,000. In Virginia, the 3 counties whose roads are not under State control received $298,000 from the State motor-fuel tax.

[5] Reimbursement to counties and local units of government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[8] To State general funds except in the following States: Alabama, county and municipal general funds; California, general funds of counties and cities, $3,776,000; Louisiana, parish general funds, $2,723,000; New Mexico, county general funds, $307,000; Wisconsin, towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles, $3,704,000. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, State parks, C. C. C. ditching, etc.; Florida, aviation; Louisiana and Massachusetts, harbor improvement; New Jersey, debt service on institutional construction bonds, $527,000, department of commerce and navigation, $328,000, other departments, $108,000; North Carolina, State probation and parole commissions; Ohio, care of indigent persons injured in motor-vehicle accidents; Pennsylvania, aviation, $244,000, cooperative work other departments, $48,000; Tennessee, debt service on nonhighway bonds; Vermont, debt service on nonhighway portion of flood relief bonds; Virginia, aviation.

[10] Includes debt service charges on emergency relief bond issues, prorated in proportion to use of proceeds for State highways, local roads and streets, and nonhighway purposes.

[11] Debt service on highway relief bonds, a State obligation incurred for improvement of local roads.

[12] Originally appropriated for relief, but later transferred by legislative action to State general fund.

[13] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle revenues to the general fund and prorated in proportion to net receipts from highway user taxes not otherwise dedicated.

[14] Due to change in registration year from January 1 to April 1.

[15] Expenditures for highway purposes have been credited against payments of motor-fuel tax and motor-vehicle revenues to the State general fund and prorated in proportion to net receipts from highway-user taxes not otherwise dedicated.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF OCTOBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,761,824	$ 876,167	49.3	$ 5,845,388	$ 2,906,953	220.1	$ 1,187,040	$ 590,890	47.5	$ 1,871,145
Arizona	1,026,203	680,107	47.7	887,592	565,761	51.3	717,770	357,416	16.6	1,008,472
Arkansas	3,523,529	1,629,159	76.7	1,785,607	886,895	78.0	467,546	233,127	26.1	288,879
California	3,584,812	1,876,157	70.2	8,014,486	4,136,279	113.4	2,609,316	1,415,170	56.4	1,347,487
Colorado	1,676,370	923,229	127.7	2,245,258	1,265,427	112.3	255,540	144,023	69.0	2,026,469
Connecticut	590,590	293,455	5.4	2,274,635	1,101,846	19.1	258,052	125,674	3.0	988,696
Delaware	95,119	47,559	2.9	1,927,523	962,982	29.6	155,407	51,044		1,048,125
Florida	943,913	471,956	25.7	3,020,271	1,499,856	91.2	840,971	420,485	17.4	2,313,581
Georgia	1,790,959	887,744	111.7	7,861,815	3,931,408	311.4	2,387,793	1,193,896	118.8	4,666,842
Idaho	1,203,714	739,131	120.2	925,031	568,325	65.1	534,222	266,364	27.2	1,345,450
Illinois	3,838,419	1,896,919	84.5	8,094,016	4,046,843	165.6	2,887,476	1,218,238	93.6	2,132,561
Indiana	2,389,929	1,191,906	61.1	7,654,635	3,677,186	129.9	2,195,394	1,065,542	32.8	890,064
Iowa	3,086,933	1,396,940	100.5	4,919,428	2,251,662	168.2	2,080,631	977,816	69.5	164,273
Kansas	2,079,065	1,038,203	152.2	6,454,040	3,225,946	427.2	2,981,227	1,490,612	181.2	2,973,201
Kentucky	1,785,343	888,277	35.0	3,487,071	1,743,535	107.4	576,781	288,390	38.0	2,907,597
Louisiana	643,730	316,311	5.4	12,506,169	3,302,037	64.7	1,377,511	681,067	37.7	2,869,293
Maine	1,003,748	496,030	20.3	1,025,511	512,755	29.3	18,200	9,100	.6	620,911
Maryland	575,000	287,500	10.2	3,463,558	1,723,362	40.1	489,303	244,651	5.1	1,233,790
Massachusetts	986,611	492,229	9.9	3,102,023	1,543,536	26.1	4,950	2,475		2,990,700
Michigan	2,363,181	1,126,875	84.2	11,128,168	5,465,734	305.7	1,053,463	471,732	35.4	284,629
Minnesota	3,256,423	1,572,971	289.3	6,599,256	3,292,517	392.4	577,421	288,711	27.1	3,135,632
Mississippi	1,231,532	508,478	43.3	6,646,274	3,006,631	333.6	2,026,160	990,330	119.7	1,295,200
Missouri	2,498,269	1,249,062	130.4	7,608,884	3,476,421	194.2	3,576,503	1,319,892	119.4	3,518,941
Montana	3,789,198	2,145,589	255.7	2,104,985	1,187,884	129.5	644,474	363,677	29.5	3,152,825
Nebraska	2,914,534	1,452,684	331.6	5,247,037	2,540,265	621.9	1,686,449	843,224	180.7	2,013,243
Nevada	1,404,260	1,208,281	74.6	1,166,186	1,015,652	48.5	9,615	8,381	.2	756,831
New Hampshire	642,060	314,046	18.4	1,220,387	599,658	27.1				904,734
New Jersey	1,126,220	562,820	8.9	4,595,830	2,297,915	32.9	1,047,230	523,615	5.5	1,610,075
New Mexico	1,327,401	817,541	123.7	1,827,559	1,109,792	92.7	343,273	206,925	7.1	970,729
New York	4,332,626	2,133,608	88.7	16,837,591	8,172,935	224.4	2,248,227	989,209	31.2	328,817
North Carolina	3,327,539	1,662,772	165.4	4,814,917	2,407,375	232.2	765,020	366,115	27.9	1,328,590
North Dakota	1,322,620	719,223	134.4	3,079,856	1,704,036	245.7	2,483,624	1,261,570	201.6	3,215,741
Ohio	1,805,773	902,886	23.8	13,736,202	6,843,772	126.5	4,005,118	2,001,379	32.8	3,709,535
Oklahoma	1,496,092	793,964	72.4	3,340,533	1,764,846	106.3	1,063,885	511,530	42.6	4,015,103
Oregon	2,668,746	1,592,001	135.0	1,962,173	1,175,844	40.6	1,328,524	613,810	36.8	625,580
Pennsylvania	2,894,188	1,419,755	40.7	12,775,559	6,344,434	117.9	2,636,166	1,305,860	25.8	2,099,685
Rhode Island	573,613	286,190	4.7	1,190,347	594,172	12.9	497,325	248,330	3.6	747,071
South Carolina	805,047	385,103	46.5	2,319,020	1,118,158	166.5	933,689	416,210	69.6	2,150,306
South Dakota	2,174,510	1,217,052	330.5	4,298,190	2,598,660	600.3	1,234,280	774,350	178.0	2,057,833
Tennessee	1,487,471	738,520	32.4	3,767,502	1,883,751	130.9	523,998	261,999	16.8	3,690,328
Texas	6,014,902	2,967,937	383.1	6,903,032	3,425,362	308.3	3,914,199	1,854,235	163.8	5,163,273
Utah	797,848	581,181	58.8	904,910	677,622	46.8	630,615	240,560	14.9	735,206
Vermont	648,589	324,295	23.3	1,119,784	558,005	29.2	250,623	125,312	5.0	272,018
Virginia	1,588,149	747,427	48.7	3,909,259	1,882,228	73.9	809,254	402,834	17.2	1,072,621
Washington	2,250,340	1,184,481	43.7	3,060,584	1,622,135	62.2	52,315	27,900	1.2	799,736
West Virginia	1,304,780	648,925	38.1	3,068,345	1,527,971	88.6	419,199	209,600	10.2	1,628,167
Wisconsin	3,602,500	1,753,281	112.9	3,567,914	1,771,735	153.9	436,907	212,894	13.1	3,240,854
Wyoming	1,378,588	868,610	162.3	945,228	600,017	95.1	487,651	311,701	58.8	667,543
District of Columbia	224,000	112,000	1.6	314,242	157,121	2.9	250,100	117,016	1.5	337,231
Hawaii	95,573	44,205	1.7	605,628	304,762	8.9	228,633	114,161	3.9	1,498,907
Puerto Rico	20,338	9,980	.2	1,704,384	843,290	27.9				717,500
TOTALS	93,952,721	48,480,722	4,425.6	227,863,873	111,823,294	7,330.4	58,189,070	28,159,042	2,321.4	91,432,020

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF OCTOBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 175,144	$ 87,363	9.4	$ 1,276,990	$ 635,975	60.7	$ 37,167	$ 18,583		$ 358,928
Arizona	97,773	49,619	10.5	249,237	179,933	8.1				231,780
Arkansas	358,381	158,811	14.6	306,339	152,958	20.6	105,323	70,125	12.2	35,478
California	502,722	275,750	22.3	600,819	322,833	19.8	165,660	88,200	2.2	739,438
Colorado				258,151	122,631	4.9	51,494	29,022	2.1	138,569
Connecticut	199,876	98,121	2.0	275,614	131,200	4.4				165,242
Delaware	69,537	34,768	7.8	98,995	41,350	7.7				268,125
Florida	12,030	6,015		509,329	254,665	3.2	398,579	173,817	17.0	194,942
Georgia	53,282	26,106	8.3	763,874	366,937	64.0	438,698	219,349	43.5	1,001,758
Idaho	98,027	60,463	17.0	190,312	114,011	10.7	40,748	24,382	3.0	104,989
Illinois	1,269,585	632,618	67.7	1,208,200	589,100	36.2	327,600	147,500	14.1	161,709
Indiana	410,671	200,658	29.1	128,482	64,241	5.6	225,336	112,600	12.2	855,868
Iowa	1,778,339	846,670	353.6	783,453	371,896	191.3	500,830	235,853	160.4	88,647
Kansas	234,574	117,287	29.6	773,916	389,866	50.7	427,439	213,719	28.1	1,109,944
Kentucky	539,652	162,935	40.9	606,853	200,095	35.0	475,019	125,751	21.3	254,327
Louisiana	41,637	20,818	3.7	256,292	128,091	22.0				458,796
Maine	213,162	101,890	12.1	126,296	61,048	6.4				5,551
Maryland	45,300	22,650	1.1	136,390	68,195	7.1	86,000	42,500	6.1	383,032
Massachusetts	123,647	61,177	2.8	508,159	252,095	10.7	37,950	18,975	.4	479,637
Michigan	867,931	425,758	61.9	854,040	427,020	76.4	536,960	268,480	41.3	339,469
Minnesota	234,436	113,490	31.1	1,021,515	510,041	154.9	425,943	212,972	55.2	761,519
Mississippi	172,962	86,481	10.6	846,952	417,976	41.9	209,800	93,365	9.3	436,082
Missouri	381,414	190,305	55.6	416,637	195,566	41.2	356,812	141,812	37.7	633,688
Montana	606,308	342,715	78.7	117,816	66,483	5.3				647,944
Nebraska	336,160	167,619	63.6	684,169	336,101	89.3	142,915	71,457	28.4	216,967
Nevada	176,020	147,191	37.1	136,697	118,675	11.4	80,414	60,944	6.8	6,976
New Hampshire	85,641	40,146	2.2	60,190	29,832	1.2				167,812
New Jersey	319,500	159,750	10.6	318,057	158,940	11.4				503,134
New Mexico	101,564	63,386	13.1	634,137	343,277	28.8				86,993
New York	1,067,601	533,801	43.9	2,202,315	1,057,353	55.6	118,500	42,000	7.0	76,719
North Carolina	597,005	298,369	58.6	501,453	251,693	39.0	59,190	29,595	5.8	298,943
North Dakota	42,880	24,583	.3	141,704	75,952	1.2	27,520	14,750	2.4	1,014,689
Ohio	510,397	255,133	18.6	2,712,130	1,354,805	87.8	403,280	201,640	17.0	676,153
Oklahoma	581,887	308,724	43.8	209,265	110,288	13.3	92,880	49,059	3.1	931,680
Oregon	330,679	179,276	53.4	247,084	103,394	16.4	162,564	75,990	13.5	181,443
Pennsylvania	877,923	431,438	33.8	1,446,009	722,096	36.1	128,000	64,000	1.9	211,587
Rhode Island	150,728	75,309	3.4	89,358	44,679	.2	88,661	44,265	.9	45,797
South Carolina	149,900	61,994	12.3	614,103	233,873	80.6	284,667	91,000	41.3	167,095
South Dakota				3,624	3,624		24,600	13,780	9.0	1,267,133
Tennessee	109,151	52,519	7.9	223,822	111,911	10.1	101,734	50,867	2.8	865,132
Texas	1,095,726	536,563	166.2	729,528	360,278	56.4	505,300	234,785	47.4	708,420
Utah	25,247	16,100	6.2	245,282	156,660	25.4				124,932
Vermont	255,700	86,334	9.5	250,608	65,721	10.0	42,494	21,247	1.2	1,253
Virginia	301,000	139,038	17.4	438,182	199,955	17.9	213,302	101,655	9.8	184,705
Washington	200,724	105,688	9.5	422,281	223,839	21.5	234,587	121,700	25.0	142,634
West Virginia	262,400	130,675	15.2	74,819	37,409	3.4	127,225	63,612	4.5	409,548
Wisconsin	236,054	117,114	3.3	698,927	349,360	26.6	79,661	35,800	1.7	615,766
Wyoming	433,925	258,391	42.8	73,067	43,669	4.0	73,555	47,016	5.1	70,147
District of Columbia	38,300	19,150	.6	92,384	45,692	.8				22,150
Hawaii	277,851	138,782	8.6	1,096	1,096					158,778
Puerto Rico				302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	17,050,353	8,469,541	1,552.3	25,867,177	12,752,018	1,551.2	7,893,595	3,699,307	702.8	19,092,456

U. S. GOVERNMENT PRINTING OFFICE 1940

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Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

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Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF OCTOBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,100	$ 4,100			1	$ 720,404	$ 700,327	6		1	$ 47,539	$ 47,539	1	1	7	$ 916,112
Arizona	192,342	184,976	3			179,037	178,688	1			6,006	6,006			2	242,091
Arkansas	85,999	85,999	1		6	1,245,833	1,241,467	12	1	1	144,553	144,553	1		9	332,818
California	238,449	238,449	3		3	828,771	650,291	2	1		507,007	505,045	6		12	1,227,650
Colorado						283,222	283,222	1			3,401	3,401			1	921,104
Connecticut	270,224	264,463	1	1		517,556	512,317	6								449,334
Delaware	65,760	65,760			12	117,498	117,498	1		4	13,839	13,839	4		1	467,605
Florida	207,524	203,025	2		2	37,626	37,626	1			194,523	193,998		1	25	1,220,136
Georgia	100,753	100,658	2		12	895,795	895,795	11	5		531,974	531,974	2	4	6	1,735,588
Idaho	206,100	202,670	5		2	8,830	8,830			1	89,237	89,237			31	413,990
Illinois	600,672	547,061	4	1	23	2,294,881	2,084,758	9	1	49	215,158	184,666			77	1,975,749
Indiana	262,468	262,468	1	1	35	877,185	877,185	6	1	38	87,075	87,075			27	991,923
Iowa	292,444	239,250	3		42	312,391	275,233	3		39	64,788	61,696			19	1,205,976
Kansas	563,185	563,185	8		11	409,294	408,816	4	1	3	122,900	122,900	3	1	11	1,157,399
Kentucky	170,478	169,935	4		11	1,037,019	1,037,019	11			35,400	35,400	1		3	549,281
Louisiana	95,496	95,496	1			345,122	291,627	2			627,884	570,153	11			804,609
Maine	157,399	156,481	1	1	5	21,934	21,934		1	1	125,990	125,990	1			251,016
Maryland	180,997	[illegible],393	1		2	476,609	444,816	2	2	5	15,600	15,600			3	767,315
Massachusetts	15,710	15,710				336,590	326,168	1			1,034	1,034			1	2,024,008
Michigan	941,190	907,876	7	1	18	1,418,695	1,418,695	3	4	5	104,950	104,950		1	18	743,016
Minnesota	766,380	758,524	8	1	6	1,254,907	1,254,907	8	4	8	213,272	213,272	1	2	6	1,009,406
Mississippi	23,760	23,760				628,634	628,634	9	1		275,000	275,000	3		3	503,012
Missouri	541,321	541,321	1	2	1	1,455,654	1,167,274	6	3		1,055,014	826,484	4	2		973,697
Montana	427,675	427,675	5		2	82,097	82,096	1			9,155	9,155			3	350,584
Nebraska	204,949	204,949		1	7	741,392	741,392	11	1	3	302,249	302,249	5		1	265,549
Nevada	11,144	11,144			7	125,527	125,527	2		6	21,199	21,199			12	115,071
New Hampshire	100,989	100,953	3			149,458	148,638	3		1						325,194
New Jersey						792,763	792,763	5		1	73,220	73,220		1	3	1,189,123
New Mexico	140,504	140,504	1		6	304,092	274,896	4								482,206
New York	428,630	427,120	3	1		3,786,319	3,718,553	9	19		244,245	240,245		1	8	2,831,215
North Carolina	164,092	164,029	1	1	13	989,429	989,189	13	5		95,530	95,530			27	818,777
North Dakota	448,315	448,150	5		15	404,220	404,220	4		6	5,700	5,700			2	528,663
Ohio	369,098	347,336	4		2	2,462,415	2,413,225	10	3	13	818,157	790,393	4		2	2,235,007
Oklahoma	253,288	252,261	2	1	24	591,587	590,240	10		1	205,628	202,212	1		43	1,880,962
Oregon	208,639	117,537	3			254,208	197,981	1			5,790	5,790			2	357,935
Pennsylvania	618,267	618,267	8	1		1,878,155	1,868,351	13	1		1,328,418	1,324,918	9	2		3,077,356
Rhode Island	3,831	3,750				192,501	192,501		1							94,451
South Carolina	325,148	325,148	4	2	7	184,294	183,891	1	1	12	162,832	162,832			31	916,503
South Dakota	72,600	72,600	1	1	2	282,902	282,042	7		2	462,535	446,585	10			739,692
Tennessee	204,265	204,265	1	2		173,264	173,264	1	1		34,462	34,462		1		1,715,788
Texas	1,145,408	1,140,600	10		3	1,181,428	1,171,063	10	1	6	77,700	77,700	1		2	1,998,980
Utah	12,419	12,208			3	37,963	37,963			10	108,747	108,640	2		40	246,680
Vermont	5,995	5,995			2	218,520	218,520		2	4	25,623	25,623			4	220,367
Virginia	81,101	80,765	1	2	3	691,669	690,339	6		2	140,842	140,842		2	10	593,601
Washington	242,856	242,856	3	1	5	123,070	121,570	1	1		198,172	198,168	1	2	7	390,610
West Virginia	5,400	5,400			2	229,982	229,982	2			66,670	64,580	2		1	1,079,957
Wisconsin	815,489	799,460	6	3	7	470,132	441,102	3	1	3	17,167	17,167			5	1,365,384
Wyoming						540,923	540,923	5			22,086	22,085	1			171,492
District of Columbia						59,061	59,061	1								150,009
Hawaii						194,036	194,028	2			9,494	9,494			2	292,509
Puerto Rico						584,007	579,336	11								414,857
TOTALS	12,272,853	11,965,132	117	24	302	33,428,901	32,325,783	241	62	225	8,917,765	8,538,601	74	21	467	45,731,357

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
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VOL. 21, No. 10 DECEMBER 1940

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PUBLIC ROADS

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Issued by the

FEDERAL WORKS AGENCY

PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 10

December 1940

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

SOIL DISPLACEMENT UNDER A LOADED CIRCULAR AREA[1]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **L. A. PALMER**, Associate Research Specialist, and **E. S. BARBER**, Junior Highway Engineer

THIS REPORT describes a procedure for evaluating the supporting characteristics of the subgrade under flexible types of pavement. The procedure uses laboratory-determined stress-deformation curves for the subgrade soil in conjunction with rational theoretical analyses. The method of approach does not include the making of penetration and loading tests directly on the subgrade, a procedure that has various disadvantages. A correlation between measured deflections of the subgrade under wheel loads and the deflections computed from laboratory test data and theory is an indicated experimental procedure that should extend knowledge in this field.

Various formulas for use in the design of flexible-type pavements have been advanced in the past, but the theories and assumptions upon which they have been based have not permitted of experimental verification. In a rational design, knowledge of the stress-deformation characteristics of the subgrade soil is essential. Such knowledge would make it possible to set up maximum allowable deformations of the base to serve as criteria for use in designing the surface.

This report presents a method of evaluating the supporting characteristics of the subgrade under flexible-type pavements, and, for the first time, suggests definite experimentation that can be made. It is felt that the suggested method has considerable promise and an extensive study is warranted to check the theory. Accordingly, a series of tests, including large-scale outdoor tests, is planned to substantiate the theory or to determine what modification may be necessary.

The system of stresses at any point within a semi-infinite, elastically isotropic body produced by a uniform load over a circular area at the surface has been determined by A. E. H. Love[2] and S. D. Carothers.[3] Formulas for the vertical displacement, V, at any point of the elastic body due to the surface load are given in various texts[4] dealing with the theory of elasticity. The use of such formulas involves knowledge concerning two elastic constants—Poisson's ratio, μ, and the modulus of elasticity, E. Test data as reported by Terzaghi[5] and others show that the deformations of soils under load are not characteristic of elastic materials. Hence, there is difficulty in applying the formulas based on the assumption of elastic properties.

The purpose of this report is to indicate a method of computing vertical displacement in soil due to a uniform load over a circular area by the use of triaxial compression test data. The vertical displacement or settlement so computed is that caused solely by lateral yield of the soil. It is assumed that this type of settlement, S_L, is completed prior to the occurrence of any settlement due to consolidation of the supporting soil. A modulus of deformation, C, is used in the formulas instead of the modulus of elasticity, E, and since it is assumed that the settlement, S_L, occurs at constant volume, μ is necessarily equal to ½.

The modulus of deformation, C, is herein defined as the ratio of stress to deformation without regard to the nature of the deformation whether it be elastic or plastic deformation or both.

The method of analysis used in this paper for the case of a uniform load over a circular area may be applied, with certain modifications, to the case of a parabolic or conical distribution of load over the circular area.

MODULUS C DETERMINABLE FROM STABILOMETER TEST DATA

In stabilometer tests, cylindrical soil samples encased in rubber sleeves are compressed to failure by applying a vertical load with or without lateral pressure. Lateral pressure is applied by air or fluid and is constant during an individual test. The decrease in length, Δh, of the sample with increasing vertical load may be measured by means of a micrometer dial attached to the moving plunger or by an automatic recording device. The initial length of the sample is designated as h.

A more complete description of plotting the stabilometer test data in connection with computing C has already been published.[6] In figure 1, the portions of the three curves for which the vertical pressure, v, is less than the lateral pressure, l, have been omitted and the coordinate axes have been moved to the right so that the point where $l=v$ falls on the $\frac{\Delta h}{h}=0$ axis.

In figure 1, the slope of any of the secant lines is taken as the modulus of deformation, C, within the range of loading indicated. The more nearly straight the line representing the v versus $\frac{\Delta h}{h}$ relationship, the more nearly the secant modulus becomes a tangent modulus. The greater the curvature of the line representing the v versus $\frac{\Delta h}{h}$ relationship, the greater is the divergence of the secant line from the curve. Usually this divergence increases with increasing load.

From the theory of elasticity, the vertical strain ϵ_z, at any point on the axis of loading in the stressed earth below the uniformly loaded circular area of radius, a, is

---

[1] Paper presented at the Twentieth Annual Meeting of the Highway Research Board, December 3, 1940.

[2] The Stress Produced in a Semi-Infinite Solid by Pressure on Part of the Boundary, by A. E. H. Love. Philosophical Transactions of the Royal Society, series A, vol. 228, 1929.

[3] Test Loads on Foundations as Affected by Scale of Tested Area, by S. D. Carothers. Proceedings, International Mathematical Congress, Toronto, pp. 527–549, 1924.

[4] See for example equations 203 and 204, page 335, of Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Company, first edition, 1934.

[5] Determination of Consistency of Soils by Means of Penetration Tests, by Charles Terzaghi. PUBLIC ROADS, vol. 7, No. 12, Feb. 1927.

[6] The Settlement of Earth Embankments, by L. A. Palmer and E. S. Barber. PUBLIC ROADS, vol. 21, No. 9, November 1940.

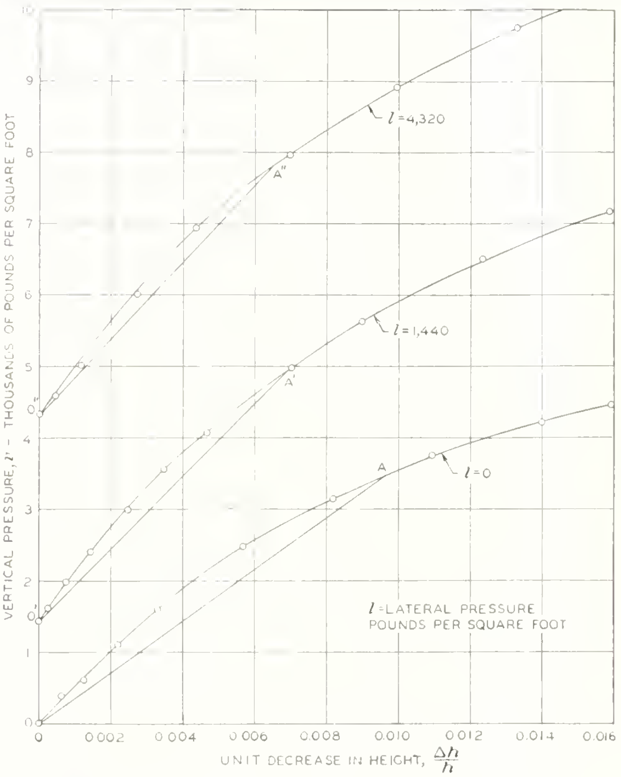

Figure 1.—Load-Compression Test Results.

$$\epsilon_z=\frac{\partial V}{\partial z}=\frac{1}{E}(p_z-2\mu p_r) \text{ - - - - - - - - - - - - - - } (1)$$

where z is the depth of the point; V is the vertical displacement at the point; and p_z and p_r are normal stresses in the vertical and radial directions, respectively, acting at the point.

If instead of E, the modulus of deformation, C, is used and taken as a constant, equation 1 becomes

$$\frac{\partial V}{\partial z}=\frac{1}{C}(p_z-2\mu p_r) \text{ - - - - - - - - - - } (2)$$

The expressions for the normal stresses, p_z and p_r, are

$$p_z=p\left[1-\frac{z^3}{(a^2+z^2)^{3/2}}\right] \text{ - - - - - } (3)$$

and

$$p_r=\frac{p}{2}\left[1+2\mu-\frac{2(1+\mu)z}{(a^2+z^2)^{1/2}}+\frac{z^3}{(a^2+z^2)^{3/2}}\right] \text{ - - - - - } (4)$$

where p is the unit surface load. The origin of coordinates is taken at the center of the circular area of earth surface. By substituting equations 3 and 4 in equation 2 and integrating between the limits z and ∞, there results

$$\text{maximum } V=\frac{p}{C}\left[(2-2\mu^2)(a^2+z^2)^{1/2}-\frac{(1+\mu)z^2}{(a^2+z^2)^{1/2}}\right.$$
$$\left.+(\mu+2\mu^2-1)z\right] \text{ - - - - - - - - - - - - - } (5)$$

By taking $\mu=\frac{1}{2}$,

$$S_L=\frac{3pa^2}{2C(a^2+z^2)^{1/2}} \text{ - - - - - - } (6)$$

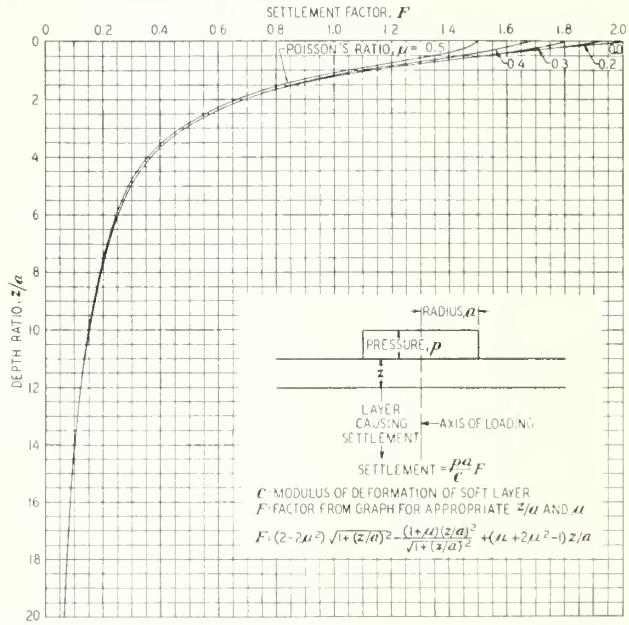

Figure 2.—Settlement Under Center of Uniform Circular Load.

or in general,

$$\text{maximum } V=\frac{pa}{C}F \text{ - - - - - - - - - - - - - - } (7)$$

where

$$F=(2-2\mu^2)\sqrt{1+\left(\frac{z}{a}\right)^2}-\frac{(1+\mu)\left(\frac{z}{a}\right)^2}{\sqrt{1+\left(\frac{z}{a}\right)^2}}+(\mu+2\mu^2-1)\frac{z}{a}$$

F may be called the "settlement factor." In figure 2, F is plotted against values of z/a for values of 0, 0.2, 0.3, 0.4, and 0.5 assigned to μ. It is observed that the value of μ has but little influence on the value of F if z/a exceeds unity.

SHAPE OF LOADED AREA HAS LITTLE EFFECT ON SETTLEMENT

In equations 5 and 6, z can have any value. For $\mu=\frac{1}{2}$ and $z=0$, equation 6 becomes

$$S_L=\frac{3pa}{2C} \text{ - - - - - - - - - - - - - - - - } (8)$$

which is the total settlement due to lateral yield from the load down to infinite depth, and it also is the downward displacement of a soil particle at the center of the circular surface-contact area. Equation 8 gives S_L along the centerline. At points removed from the centerline, the settlement is less than this value. According to Timoshenko[4] the average settlement under the uniformly loaded circular area, which is the average deflection of all points over the circular contact area, is 85 percent of the maximum deflection at the center. Timoshenko[4] also shows that the average settlement under a uniformly loaded circular area is practically the same as that under a uniformly loaded square area. He shows further that the average settlement under a uniformly loaded rectangular area having sides of ratio 2:1 is about 4 percent less than that realized in the case of the circle or square. Thus the effect of shape of loaded area on settlement is not so important as might at first be thought.

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[4] Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Co., first edition, 1934, pp. 338 and 339.

The depth, z, figure 2, may be considered as the thickness of an incompressible yet flexible layer, infinite in lateral extent, below which there is yielding soil of infinite depth. Such a condition may be realized for practical purposes if a bed of stabilized soil-aggregate is spread over a clay soil and a load is applied to the stabilized surface. The settlement in this case would be due entirely to yielding of the clay if the thickness, z, of the soil-aggregate bed is assumed to remain constant under load and to have no flexural strength. Then, as shown in figure 2, for given values of p and C, increasing the ratio, z/a, either by increasing z or decreasing a, reduces the settlement factor, F, and hence the value of S_L.

This reasoning is based on the simplifying assumption that the bed of material of thickness z and the more yielding material below it comprise a single homogeneous and isotropic mass of material insofar as the system of stresses is concerned. As pointed out previously,[7] a very similar assumption has been made and used to very good advantage in estimating the settlements of structures supported by one or more layers of compressible soil with one or more intervening layers of sand.

For a uniform load on a circular area at the surface, the maximum shearing stress at each point on the axis of loading is

$$s_{max} = \frac{1}{2}(p_z - p_r) \qquad (9)$$

where the difference, $p_z - p_r$, is the principal stress difference.

From equations 3 and 4,

$$p_z - p_r = p\left[\frac{1-2\mu}{2} + (1+\mu)\frac{z}{(a^2+z^2)^{1/2}} - \frac{3z^3}{2(a^2+z^2)^{3/2}}\right] \qquad (10)$$

$$= pf$$

where

$$f = \frac{1-2\mu}{2} + \frac{(1+\mu)\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}} - \frac{3}{2}\left(\frac{\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}}\right)^3$$

Values for the principal stress differences at different depths on the axis of loading and for different values of μ are shown in figure 3. It may be noted from this figure that for $\mu = 1/2$, the maximum principal stress difference is equal to $0.58p$ and occurs at the depth, $z = 0.71a$.

It is necessary to bear in mind that $p_z - p_r$ is the difference between the vertical and lateral pressures on the axis of loading. Obviously there could be no lateral yielding of the supporting soil if p_z and p_r were equal in magnitude and of the same sign at all points.

In figure 1, the points A, A′, and A″ were selected so as to give $v - l$ the same value. In general, the modulus, C, obtained from the slopes of secant lines, such as OA, O′A′, etc., vary somewhat with the magnitude of the lateral pressure, l, maintained constant during a single test. Usally the value of C is lowest for the curve, $l = 0$. The procedure in this paper is to use an average value of C obtained from two or more curves of the type shown in figure 1 for a definite value of $v - l$, applicable to the particular problem. The value of $v - l$ which determines the average C to be used is the maximum principal stress difference at any point within the yielding soil mass and on the axis of loading. Obviously this procedure is on the side of safety.

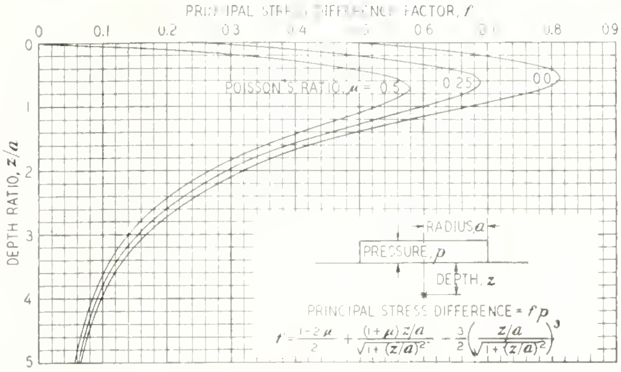

FIGURE 3.—PRINCIPAL STRESS DIFFERENCE UNDER CENTER OF UNIFORM CIRCULAR LOAD.

EXAMPLE ILLUSTRATES USE OF PRINCIPLES

The use of these principles may be illustrated by an example.

Assume the existence of a clay layer extending from the earth surface to an infinite depth. Assume there is a uniform load at the surface of 3 tons per square foot distributed over a circular area having a radius of 10 feet. Taking μ as ½, the greatest principal stress difference on the axis of symmetry is at a depth of $0.71a = 0.71 \times 10 = 7.1$ feet. Soil samples taken at this depth have the stress deformation characteristics shown in figure 1. The greatest principal stress difference $= 0.58p = 0.58 \times 6,000 = 3,480$ pounds per square foot.

On the curve, $l = 0$, figure 1, for $v - l = 3,480$, $v = 3,480$ pounds per square foot. This is at point A on this curve. The secant line, OA is drawn.

The slope of OA is $\frac{3,480}{0.0096}$ or 362,000 pounds per square foot, the value of C from this curve. For the curve obtained with $l = 1,440$ pounds per square foot in the triaxial test, the point A′ is determined by adding 1,440 to 3,480 to obtain a value, $v = 4,920$ pounds per square foot. The corresponding percentage deformation is 0.68. Then C from this curve is $\frac{3,480}{0.0068}$ or 512,000 pounds per square foot. Similarly, from the secant line O″ A″ for the curve, $l = 4,320$ pounds per square foot, C corresponding to $v = 7,800$ pounds per square foot on the curve, is computed as $\frac{3,480}{0.0065}$ or 535,000 pounds per square foot. The average modulus is then 1/3 (362,000 + 512,000 + 535,000) or 470,000 pounds per square foot.

In this problem, the depth of a layer or zone wherein compression due to yielding does not occur is zero and hence z/a is zero. From figure 2, for $z/a = 0$ and $\mu = 1/2$, $F = 1.5$. Then

$$S_L = \frac{pa}{C}F = \frac{6,000 \times 10}{470,000} \times 1.5$$

$= 0.19$ foot, or about 2.3 inches.

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[7] Stresses Under Circular Loaded Areas, by L. A. Palmer. Proceedings of the Highway Research Board, vol. 19, 1939.

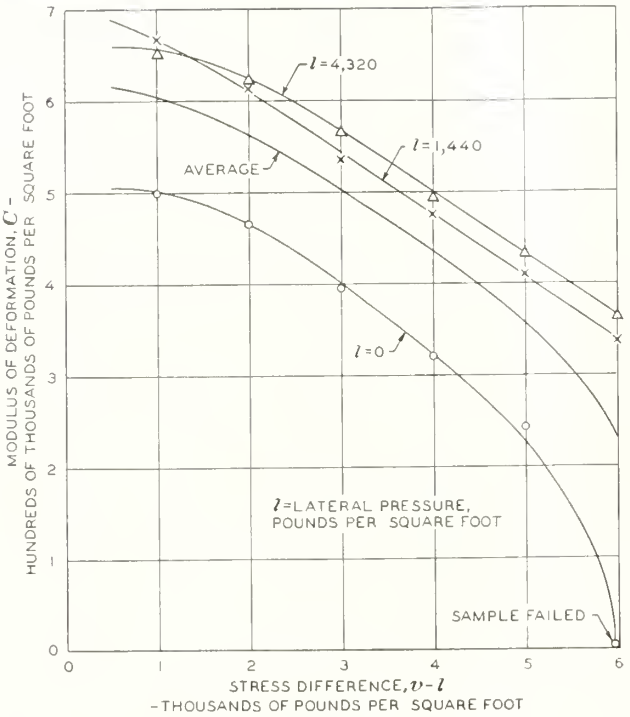

FIGURE 4.—VARIATION OF MODULUS OF DEFORMATION WITH STRESS.

Suppose now that there is a bed of compact sand 5 feet thick at the surface and that the clay is below. Neglecting the lateral displacement or vertical compaction of the sand, with $z/a=\frac{5}{10}=0.5$, the displacement factor, figure 2, is seen to be reduced from 1.5 to 1.33 and the value of S_L is reduced to $2.3\times\frac{1.33}{1.5}=2.0$ inches. Similarly, if the bed of sand is 10 feet thick, z/a becomes 1 and F, figure 2, is 1.06.

With a bed of compact sand 10 feet thick, the greatest principal stress difference is in the sand and not in the clay. The greatest principal stress difference in the clay is then 10 feet down from the ground surface on the axis of symmetry and, from figure 3, its value at the point, $z/a=1$, is $0.53p$. For the soil data plotted in figure 1, the moduli of deformation corresponding to various $v-l$ values are shown in figure 4 for the three curves, $l=0$, $l=1{,}440$, and $l=4{,}320$ pounds per square foot. An average for these three curves is also shown. For $p=6{,}000$ pounds per square foot and a maximum principal stress difference of $0.53\ p=0.53\times6{,}000=3{,}180$ pounds per square foot, the corresponding C from the average curve, figure 4, is 490,000 pounds per square foot. Taking $F=1.06$ corresponding to $z/a=1$ and $\mu=\frac{1}{2}$,

$$S_L=\frac{6{,}000\times10}{490{,}000}\times1.06=0.13 \text{ foot or } 1.6 \text{ inches.}$$

This is approximately 70 percent of the settlement without the sand, assuming no lateral displacement or compaction of the sand.

Actually, the sand would undergo some lateral displacement or compaction or both but to a considerably lesser extent than would a soft clay. Greatest settlements due to lateral yield are to be expected in plastic soils in which pore pressures of considerable magnitude are developed by loading.

With reference again to figure 4, consider a wheel load producing 10,000 pounds per square foot pressure on the surface of the clay, assuming a balloon tire and an equivalent radius of 6.5 inches for the contact area. Here the greatest principal stress difference on the axis of loading is $0.58\times10{,}000$ or 5,800 pounds per square foot, corresponding to an average C value, figure 4, of 270,000 pounds per square foot. For $z/a=0$, $F=1.5$ and

$$S_L=\frac{10{,}000\times6.5}{270{,}000}\times1.5=0.36 \text{ inch.}$$

Suppose now that the wheel rests on a flexible-type pavement of thickness 6.5 inches and that the lateral movement and compaction of pavement material under the wheel load is negligible. Under the pavement is the same clay, extending to an infinite depth. The maximum principal stress difference in the clay is $0.53p$ for $z/a=1$ and $\mu=\frac{1}{2}$, and is $0.53\times10{,}000=5{,}300$ pounds per square foot. The corresponding C value, average curve, figure 4, is 330,000 pounds per square foot and $F=1.06$, figure 2. Then

$$S_L=\frac{10{,}000\times6.5}{330{,}000}\times1.06=0.21 \text{ inch.}$$

By making similar computations for the same soil and for various values of z, the settlements shown in table 1 are obtained. It is observed in table 1 that when the pavement thickness is of magnitude such that z/a is equal to or greater than 2, any further increase in pavement thickness effects a relatively small decrease in S_L. It is also noted that the modulus, C, increases rapidly as the principal stress difference decreases, a fact that is indicative of the curvilinear relationship between v and $\frac{\Delta h}{h}$.

TABLE 1.—*The effect of pavement thickness on the estimated settlement, S_L, due to yielding of the subgrade. Unit load=10,000 pounds per square foot, equivalent radius of wheel load=6.5 inches and $\mu=\frac{1}{2}$*

Thickness of pavement, z	$\frac{z}{a}$	Maximum principal stress difference in subgrade, $v-l$	Modulus of deformation, C	Deflection factor, F	Settlement, S_L
Inches		*Pounds per square foot*	*Pounds per square foot*		*Inches*
0	0	5,800	270,000	1.50	0.36
3.25	0.50	5,800	270,000	1.33	.32
6.50	1.00	5,300	330,000	1.06	.21
8.00	1.23	4,600	390,000	.93	.16
10.00	1.54	3,750	455,000	.80	.11
12.00	1.85	2,950	505,000	.71	.09
15.00	2.31	2,200	550,000	.59	.07
18.00	2.77	1,600	580,000	.51	.06

The effect of moisture content and compaction on the modulus, C, of a typical clay soil is shown in table 2. The settlement, S_L, caused by lateral displacement of this clay when subjected to a uniform load over a circular area is also shown. For the computations shown in table 2, μ is taken as $\frac{1}{2}$ and the surface load is assumed to be applied directly to the clay; that is, $z/a=0$.

The effect of moisture content and density is strikingly illustrated by the computed values in table 2.

(Continued on page 198)

EFFECTS OF HIGHWAY LIGHTING ON DRIVER BEHAVIOR

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by W. P. WALKER, Assistant Highway Engineer-Economist

Limited data indicate that the behavior of drivers operating under artificial light conforms very nearly to their behavior in the daytime, but that the behavior of drivers at night without overhead lights differs measurably from their behavior in the daytime. Differences between the behavior of drivers during daylight and darkness are most apparent in the frequency of passing and in the transverse positions of vehicles on the pavement. There is inconclusive evidence that speed may also be affected.

During daytime the drivers utilized 57.7 percent of the available opportunities for passing as compared to 55.6 percent during nighttime with the highway lighted. At night with the highway unlighted the drivers utilized only 38.5 percent of the available opportunities for passing.

The frequency distributions of transverse positions are almost identical for conditions of daytime and nighttime with the highway lighted, but there is a marked difference in these distributions for conditions of daytime and nighttime with the highway unlighted. The average position of the right wheel of passsenger cars moving freely was 3.3 feet from the edge of a 20-foot pavement during both daytime and nighttime with the highway lighted. With the highway unlighted, this average position was ½ foot nearer the center of the road.

ACCIDENT RECORDS have consistently shown that nighttime driving is more hazardous than daytime driving. During recent years illumination engineers have performed considerable research on highway lighting with the object of reducing the ratio of nighttime to daytime accidents per vehicle-mile of travel. Several hundred miles of rural highway are now lighted, many of these being temporary installations for purposes of demonstration and experimentation. One such installation is a 1-mile section on U. S. Route 422 near Chagrin Falls, Ohio. Excessive grade and curvature at this location result in its being considered highly hazardous, and for this reason it was selected by the Nela Park Engineering Department of the General Electric Company for study of methods of illumination and measurement of factors affecting visibility on lighted roads.

In the fall of 1939, the Public Roads Administration and the Ohio Department of Highways conducted studies at this location in an effort to determine the effect of lighting on driver behavior. These studies were made over a 5-day period and three types of equipment were employed, each designed to obtain different information regarding driver behavior. With this equipment, comprehensive data were collected on passing practices, transverse positions of vehicles, and vehicle speeds and spacings. The primary objective was to determine to what extent driver behavior varied in daytime, in nighttime with the road lighted, and in nighttime with the road unlighted. Inclement weather during the observations increased the number of variables to include conditions of wet and dry pavement.

The highway approaches the section on a tangent with a slightly undulating grade. About ¼ mile from the first light and within the lighted section, the road rises slightly, then drops sharply on a grade of about 10 percent for a distance of approximately ½ mile. There are two horizontal curves on this grade, one of them being very sharp. The lower ¼ mile of the section is approximately level tangent. Figure 1 is a sketch of the plan and profile showing the operating positions of the study equipment.

The surface is portland-cement concrete pavement in fairly good condition, having a width of 20 feet except on and below the hill where it is 27 feet wide. The shoulders on the 20-foot section are 10 feet wide and consist of 2 feet of clay-gravel and 8 feet covered with grass and in good condition. The section 27 feet wide has a 6-inch curb on each side. Lighting is by means of incandescent lamps in specially designed reflectors mounted 25 feet high, 125 feet apart, and extending 5 feet out over the pavement. The lights are along only one side of the road. Figure 2 shows a portion of the section in the daytime and when lighted at night.

In evaluating the effects of highway lighting from a safety standpoint the accident record itself would be the most desirable index, but, since the number of accidents per mile of highway is relatively small, to obtain reliable data for a 1-mile section of highway would require years. Moreover, no accident records were available for this particular section of road. As a substitute for accident records it is possible, by a critical examination of driver behavior under the various conditions, to judge the probable effects of these conditions. From the results of driver-behavior studies, it is possible to find instances where a driver on an unlighted highway was unquestionably driving too fast or in an otherwise reckless manner, but there is no way of proving that the particular driver might not drive just as recklessly during daylight or on a lighted highway. However, the chances are that there will be just as many reckless drivers using the highway at night when it is not lighted as when it is lighted.

A differentiation between safe and unsafe driving practices under any set of driving conditions is difficult to make. Undoubtedly, the safest driving conditions exist during hours of daylight with a dry pavement,

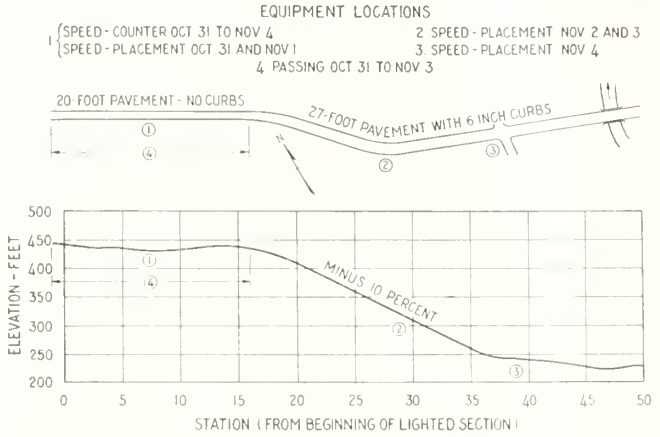
EQUIPMENT LOCATIONS
1 SPEED - COUNTER OCT 31 TO NOV 4
SPEED - PLACEMENT OCT 31 AND NOV 1
2 SPEED - PLACEMENT NOV 2 AND 3
3 SPEED - PLACEMENT NOV 4
4 PASSING OCT 31 TO NOV 3
20-FOOT PAVEMENT - NO CURBS
27-FOOT PAVEMENT WITH 6 INCH CURBS
N
①
④
②
③
ELEVATION - FEET
500
450
400
350
300
250
200
MINUS 10 PERCENT
0
5
10
15
20
25
30
35
40
45
50
STATION (FROM BEGINNING OF LIGHTED SECTION)

FIGURE 1.—PLAN AND PROFILE OF ONE-MILE SECTION OF LIGHTED HIGHWAY ON U. S. ROUTE 422 NEAR CHAGRIN FALLS, OHIO.

and the average driver under these conditions might be expected to perform in a somewhat different manner with respect to speed, distance from car ahead, and transverse position on the pavement than he would on the same highway after dark. If the drivers perform on a lighted highway at night in the same manner they perform on the highway in daylight, it is safe to assume that the vehicles are moving with greater safety and facility than they would on an unlighted highway. The degree to which driver performance on the lighted highway at night approaches that for daylight hours should be a measure of the effectiveness of the lighting in bringing about safer driving conditions.

PASSING PRACTICES STUDIED ON LIGHTED SECTION

Equipment for determining the passing practices of motor-vehicle drivers has been developed and its use described.[1] [2] The equipment permits determination of the speed and time spacing of each vehicle at any point within a half-mile section, and shows whether the vehicle was in its own or the opposing lane of traffic or was straddling the centerline of the road. It does not permit a determination of the pavement edge clearances of vehicles.

Because of topographic conditions on the section of road studied, it was practical to install only two-thirds of the detector tubes, the location selected being on the tangent at the top of the hill. On a portion of this study section passing was restricted by inadequate sight distance. Because of this and the low traffic volumes prevailing, the number of passing maneuvers recorded was not great. The equipment was operated during afternoons and evenings until about 10 p. m. for 4 days. The lights were off on alternate evenings.

Table 1 shows the actual operating time of the recorders, the number of vehicles, and the number of passing maneuvers recorded under the various conditions studied. Of the 107 passing maneuvers recorded, a complete record was obtained of only 53, the other 54 having been started before entering or completed after leaving the study section. In all 107 cases, however, the passed and passing vehicles were recorded while abreast of each other so that data were obtained on at least half of each maneuver.

TABLE 1.—*Results of studies using passing equipment*

Condition	Net hours studied	Vehicles recorded	Passings recorded
Daylight	1. 85	448	41
Night—lights on	3. 77	496	20
Night—lights off	4. 48	616	20
Twilight	1. 50	462	26
Total	11. 60	2, 022	107

Conclusive results obviously cannot be drawn from such a small and varied sample. It is of interest, however, to examine a few of the passing maneuvers that were made under what might be considered hazardous conditions. Since no passing maneuver was made when an oncoming vehicle was so near as to constitute a hazard, this sample includes only passing maneuvers that were made where the driver could see less than 400 feet of road surface ahead of him. This figure is used because it represents the sight distance at the point of beginning of a double white line center marking. The remainder of the passing study section had no centerline marking except the black center joint.

Eighteen vehicles started to pass where the sight distance was 400 feet or less and these maneuvers are shown graphically in figure 3. These data are presented merely as a matter of general interest, since any comparison in numbers would be inconclusive and the similarity of the passing maneuvers permits of little differentiation for various conditions. It will be noted that, under all four conditions of light, there are cases where the passed and the passing vehicles were abreast of one another at points where the sight distance was only 200 feet. The speeds of these passing vehicles varied between 25 and 50 miles per hour. Had an oncoming vehicle made its appearance during one of these maneuvers, either the passing or oncoming vehicle would almost certainly have been required to take refuge on the shoulder to avoid a collision. In these 18 passing maneuvers, 11 of the passed vehicles were trucks, busses, or tractor-semitrailer combinations moving relatively slowly.

Several of the passing maneuvers of westbound vehicles (fig. 3) were accomplished without creating any traffic hazard, the reason being that the sight distance increased to about 2,000 feet before the passing vehicle was completely in the left lane. These were violations of the center striping that would not be so classified had the marking been of the directional type which permits passing in one direction while prohibiting it in the opposite direction. The number of vehicles that could have passed but were discouraged from doing so by reason of the center striping cannot be determined.

Table 2 shows the relationship between the actual number of passing maneuvers recorded and the number that could have been accomplished under favorable conditions. Classified as "potential" passings are those cases where a vehicle was following another vehicle at a spacing of 1½ seconds or less at a point where no restriction was offered to passing by an oncoming vehicle within 1,500 feet or by a sight distance less than 1,200 feet. The percentage that these potential passings are of the total is shown in the last column. These figures show that 42.3 percent of these drivers were reluctant to pass during daylight as compared to 61.5 percent at night when the highway was unlighted. When the highway was lighted, however, only 44.4 percent of the drivers preferred to follow the vehicle ahead rather than

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[1] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by E. H. Holmes, PUBLIC ROADS, vol. 19, No. 11, January 1939.

[2] Progress in Study of Motor-Vehicle Passing Practices, by O. K. Normann, PUBLIC ROADS, vol. 20, No. 12, February 1940.

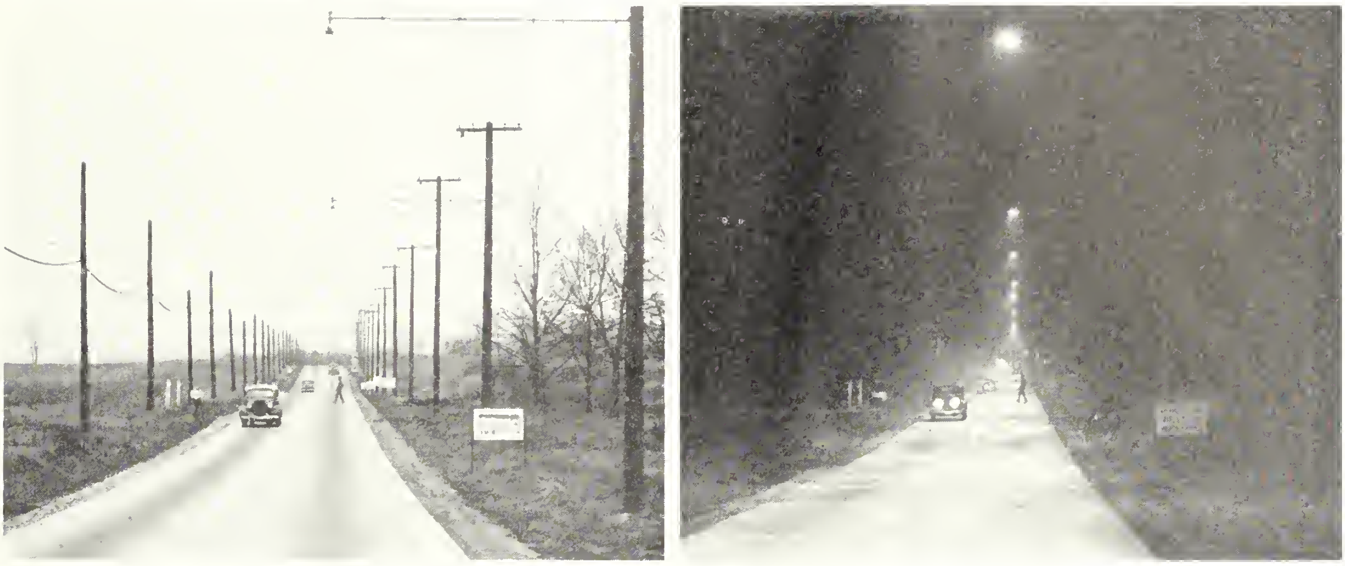

Figure 2.—Appearance of a Portion of the Section in Daylight and When Lighted at Night.

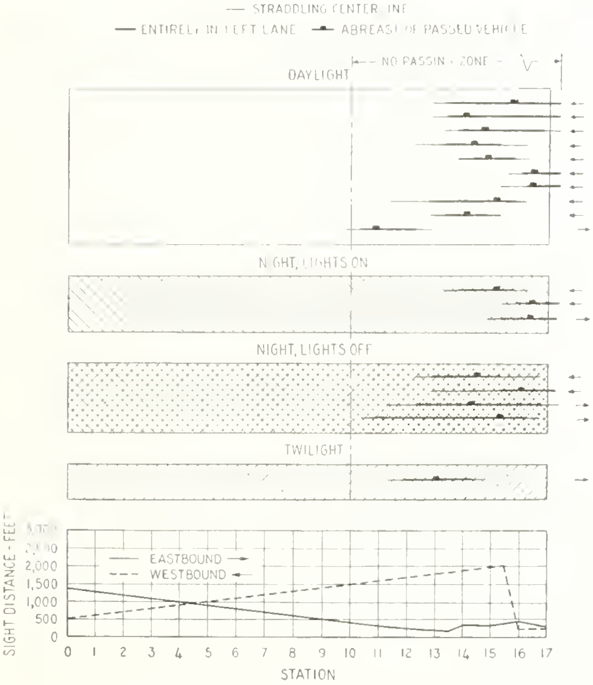

Figure 3.—Passing Maneuvers Made in Violation of Center-Striped No-Passing Zone.

pass it. This compares favorably with the data for daylight conditions. It might be concluded from this that the drivers using the highway while it was unlighted were more cautious than those using the highway while it was lighted, but the important fact shown here is that the driving practices observed while the lights were on conformed much more nearly to those for daylight conditions than did the driving practices on the unlighted highway.

SOME HAZARDOUS DRIVING FOUND UNDER ALL CONDITIONS OF LIGHTING

The passing study equipment is well adapted to studying the variations in speed of vehicles over a length of highway. Such an investigation is of interest here to determine what effect the combination of a large diamond-shaped "Hill" sign and a flashing danger signal, both located near the crest of the rise just preceding the steep descent, had upon the speeds of vehicles under various conditions. In figure 4, the average speeds of vehicles as maintained throughout the section are plotted. The warning signs were located opposite station 14, facing eastbound traffic.

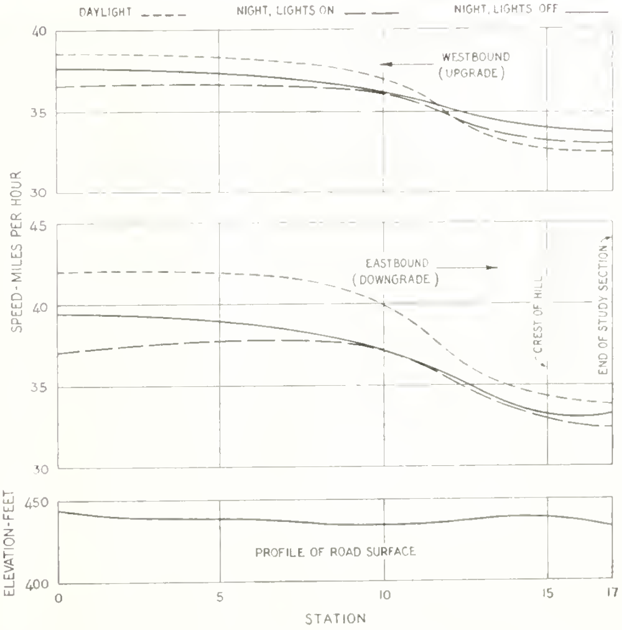

Figure 4.—Average Speeds of All Vehicles as Recorded by Passing Equipment.

Table 2.—*Relations between the actual and potential number of passings under various conditions*

Condition	Number of passings			Percentage that potential is of total
	Actual	Potential	Total	
Daylight	41	30	71	42.3
Night—lights on	20	16	36	44.4
Night—lights off	20	32	52	61.5
Twilight	26	25	51	49.0

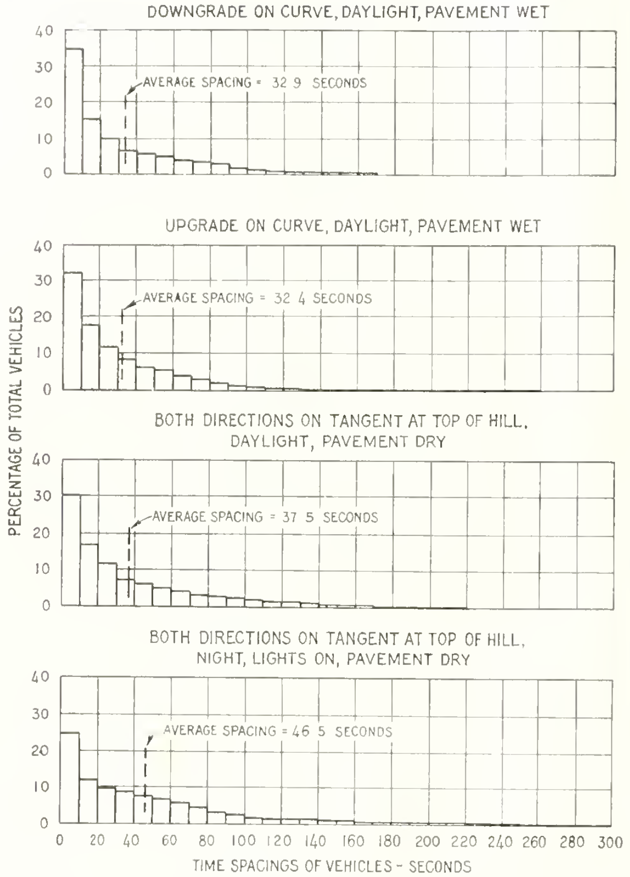

FIGURE 5.—FREQUENCY DISTRIBUTIONS OF TIME SPACINGS.

The speed curves for eastbound vehicles vary considerably for the three conditions shown. The average speed of the vehicles on the lighted highway showed a slight increase for the first 850 feet, whereas with the lights off, the average speed began to decrease almost immediately after entering the section. A number of explanations of this difference in behavior suggest themselves but none has any plausible basis. For each condition of lighting, the average speed showed a noticeable decrease upon approaching the warning signs, the amount of decrease varying from 5 miles per hour when the highway was lighted to 8 miles per hour during daylight. Without these warning signs, drivers unfamiliar with the road would be unaware of approaching any danger since the terrain visible from this point did not reveal the hill.

The ordinate of figure 4 for westbound vehicles represents their average speeds as they ascended the grade. The normal speed of these vehicles was somewhat lower than for the eastbound traffic.

The results of the passing study show that there are certain drivers whose hazardous driving habits cannot be corrected by means of artificial lighting, since such drivers are present under all conditions of light and darkness.

The results further show that there is a marked difference between the normal behavior of drivers during daylight and darkness, and that the behavior of drivers under artificial light conforms more nearly to their behavior during daylight than it does to their behavior during darkness.

The effect of weather conditions on driver behavior was more noticeable in the results of the speed-placement study than in the passing study. For this reason the results for various lighting conditions have been further classified to show variations caused by wet pavement.

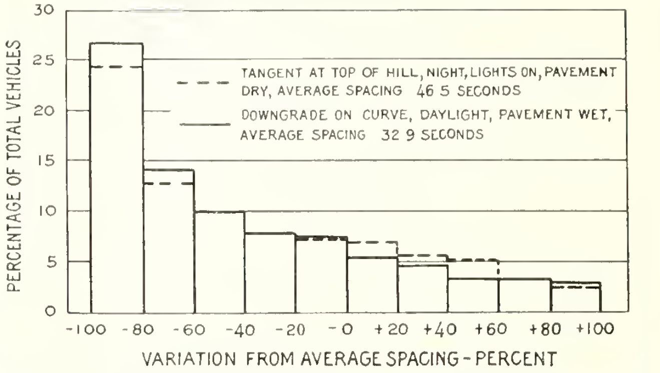

FIGURE 6.—FREQUENCY DISTRIBUTION OF VARIATIONS IN TIME SPACINGS FROM THE AVERAGE SPACING.

The passing equipment and placement equipment are two distinct sets of apparatus and are operated independently of each other. Speed and time-spacing data for all vehicles are an incidental part of the passing study records, and are available for any point within the study section. These data are also obtained by the speed-placement recorders but at only one point. In addition, the latter equipment records the positions of vehicle wheels with respect to the edge of the pavement.

Table 3 shows how the 32.49-hour net operating time of the speed-placement equipment was distributed with respect to weather and lighting conditions. This was the total time of study at three locations: One on the level tangent at the top of the hill, one on the sharp curve about midway of the steep grade, and one on the level tangent at the foot of the hill.

TABLE 3.—*Net hours of operation and vehicles recorded in speed-placement study*

Condition	Hours studied	Vehicles recorded
Daylight:		
Wet pavement	2.00	441
Dry pavement	10.13	2,388
Night—Lights on:		
Wet pavement	3.72	547
Dry pavement	5.92	1,131
Night—Lights off:		
Wet pavement		
Dry pavement	6.92	1,027
Twilight	3.80	1,225
Total	32.49	6,759

The frequency distribution of time spacings was investigated as a possible index of driving habits under various conditions of lighting and alinement. It is of interest that the time-spacing patterns varied only slightly from patterns found in previous studies, confirming the results of nearly all earlier studies. Under all conditions the percentage of vehicles traveling at or below the average time spacing was between 63 and 67. Earlier studies had showed invariably that approximately two-thirds of the vehicles traveled at or less than the average time spacing.

In figure 5, the frequency distributions of time spacings are shown for four conditions. Distributions for other conditions could also be shown but the similarity is so pronounced that further illustration is unnecessary.

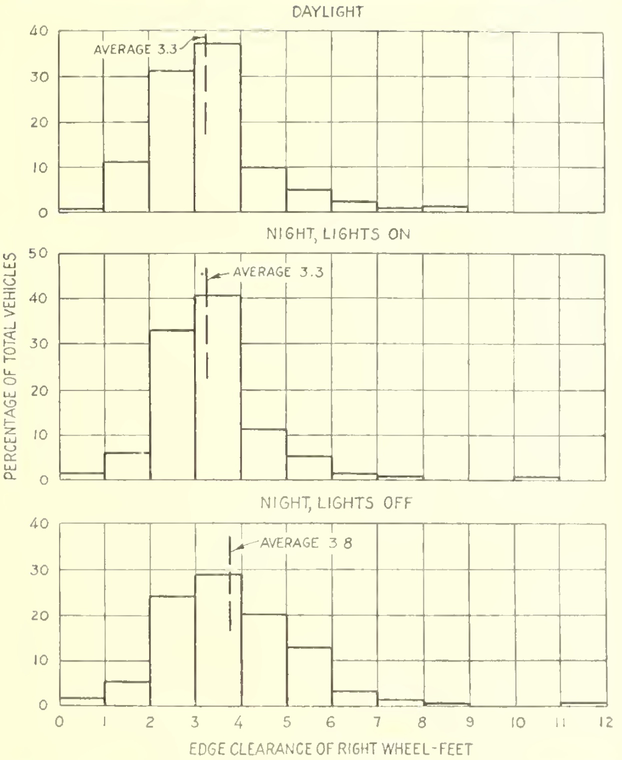

Figure 7.—Edge Clearances of Passenger Cars Moving Freely on Tangent at Top of Hill, Pavement Dry.

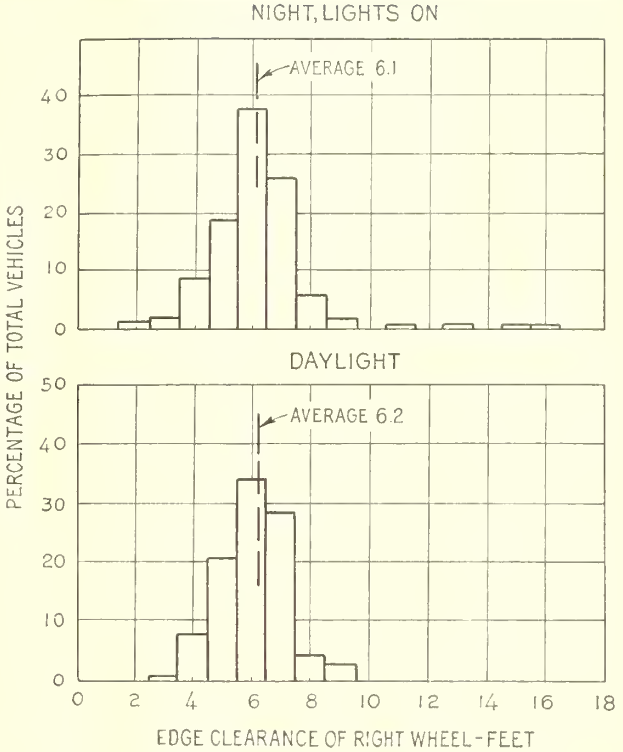

Figure 8.—Edge Clearances of Passenger Cars Moving Freely on Tangent at Foot of Hill, Pavement Dry.

The fact that the frequency distribution of time spacings is a definite function of the average spacing and hence of the traffic volume, is more apparent in figure 6, where the distribution is based on the percentage of the average spacing. Here the distribution of spacings for one condition is superimposed on that for another up to twice the average spacing. These two conditions represent the extremes in traffic volumes studied. These results show that for the traffic volumes studied the time spacing of vehicles is independent of alinement, weather, and light conditions.

PLACEMENT OF VEHICLES UNDER VARIOUS CONDITIONS COMPARED

Data on the average placement of all vehicles with respect to the edge of the pavement are useful in comparing driver behavior under various conditions. In order to eliminate insofar as possible all extraneous factors, however, the most significant placement data are those obtained while drivers were uninfluenced by the presence of a preceding or an opposing vehicle. The edge clearances of such "freely" moving passenger cars on the tangent at the top of the hill, figure 7, show frequency distributions for conditions of daylight that are very similar to those at night with the highway lighted. In both cases the average edge clearance was 3.3 feet.

The distribution of edge clearances at night with the highway lights off, however, follows a noticeably different pattern and the average placement is ½ foot nearer the center of the road. With the highway lights on, 75 percent of the drivers followed a path not more than 2 feet wider than the car, the right wheel always being between 2 and 4 feet from the pavement edge. When the highway lights were off, the same percentage of drivers had a 3-foot variance in their path, the position of the right wheel being between 2 and 5 feet from the pavement edge. On the tangent at the foot of the grade the similarity between the placements during daylight and at night with the highway lights on is almost as striking (fig. 8). The difference in the average placement for these two conditions is only 0.1 foot. No record was obtained at this location at night with the highway unlighted.

Because of differences in weather conditions no direct comparison can be made of placement data recorded on the curve. The pavement was wet when studied with the highway lights on, and dry when studied with the lights off. Furthermore, the paths of vehicles traveling upgrade were restricted by the natural tendency of of drivers to hug the inside of the curve regardless of weather or lighting conditions. For the drivers traveling downgrade there is greater freedom in selecting the path which the driver feels is consistent with safety and comfort. Figure 9 shows that, for vehicles traveling downgrade, there was a slight difference between the placement distributions at night with the highway lights on and in the daytime under similar weather conditions. However, when the highway lights were off there was a marked difference in the distribution when compared to that for daylight with dry pavement.

As mentioned previously, the speeds of all vehicles were obtained as an incidental feature of the passing study at the top of the hill, and at three points with the speedmeter: The top of the hill, below the hill, and on the grade at the curve.

From the results of the passing study, figure 4, it appears that the speeds of vehicles, particularly of those just entering the lighted section, may not be representative of normal driving practice on lighted highways. This may be caused by the fact that the passing-study section was located at one end of the lighted portion of the highway, and drivers entering the section had had

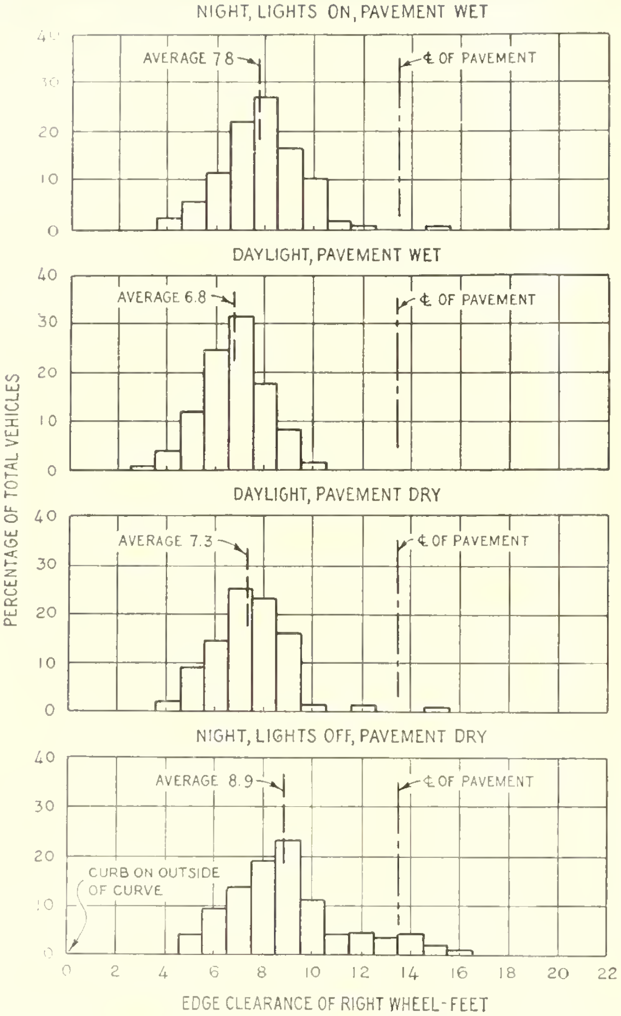

Figure 9.—Edge Clearances of Passenger Cars Moving Freely Downgrade on Curve.

no opportunity to adjust their driving to the changed condition. This assumption appears reasonable since the speeds of westbound vehicles were fairly uniform, as shown in figure 4.

At the first location of the speedmeter, 700 feet from the end of the lighted section, this same effect could be expected to influence the speed distribution. In addition, during study at this station the sample obtained under each condition was too small to indicate reliably the effect of illumination on speed distribution. The results obtained at the other two stations, located nearer the center of the lighted section, are not subject to these limitations.

Figure 10 shows that the distribution of speeds on the tangent below the hill was about the same under illumination as during daylight. The average speeds were 43 and 44 miles per hour, respectively. The posted speed limit on this road was 35 miles per hour, but these speed distributions show that 82 percent of the vehicles traveled in excess of this speed, both during daylight and at night with the highway lighted. Under both conditions 20 percent of the vehicles traveled in excess of 50 miles per hour.

On the curve, speed distributions were more varied in character, as shown in figure 11. For vehicles going downgrade the greatest similarity in speeds seems to exist between daylight with wet pavement and night without lights but with dry pavement. The speed distribution for vehicles on lighted, wet pavement seems to be in a class by itself, the average speed of 26 miles per hour being 6 miles per hour less than for daylight under similar weather conditions. For vehicles going upgrade, there are no marked differences in the patterns of speed distribution for the various conditions, but it is of interest that the average speeds of vehicles going upgrade under the various conditions are almost identical with the speeds of vehicles going downgrade under those same conditions.

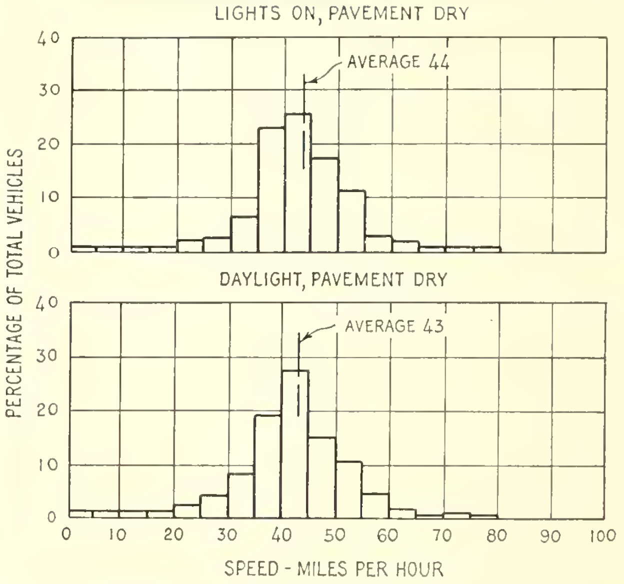

Figure 10.—Frequency Distributions of Speeds on Tangent at Foot of Hill.

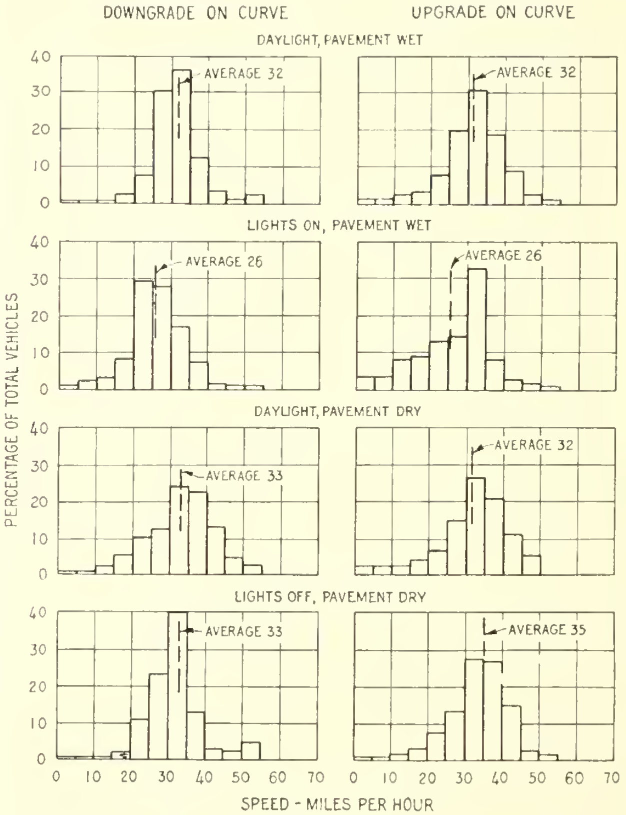

Figure 11.—Frequency Distributions of Speeds of All Vehicles.

(Continued on page 199)

EFFECT OF THE CHEMICAL PROPERTIES OF SOIL FINES ON THE PERFORMANCE OF SOIL-AGGREGATE MIXTURES

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **PAUL RAPP**, Associate Chemist, and **JACOB MIZROCH**, Junior Chemical Engineer

AN ACCUMULATION of data from laboratory tests and field observations during the past several years has shown a general relation between the physical properties of soils and their performance in base and surface courses. The sampling and laboratory test procedures used in determining the physical properties of soils are presented in detail in the Standard Specifications for Highway Materials and Methods of Sampling and Testing of the American Association of State Highway Officials. The specifications [1] for base- and surface-course materials place special emphasis on the liquid limit, plasticity index, and mechanical analyses of the soil mortar or soil fines.

In general, these physical tests have been a satisfactory means of determining the suitability of soil-aggregate mixtures for road-building purposes, but in certain instances they have not been adequate for purposes of differentiation between satisfactory and unsatisfactory materials. The purpose of this paper is to indicate the additional information that may be derived from a study of the chemical properties of the soil fines which, together with the knowledge of physical properties, provides a more adequate basis for determining suitability than do physical properties alone.

CHEMICAL PROPERTIES OF SOILS DEPENDENT ON CLAY MINERALS PRESENT

The chemical properties of soils are dependent directly on the kind and amount of clay minerals present. Hence a brief resumé of the chemistry of clay minerals is given.

Hendricks and Alexander [2] have reported that clay minerals may be included in three general classes or groups as follows:

1. The kaolin group:
    - a. Kaolinite
    - b. Nacrite
    - c. Dickite
    - d. Halloysite

    composition $Al_2O_3 . 2\ SiO_2 . 2\ H_2O$.
2. The montmorillonite group:
    - a. Montmorillonite
    - b. Beidellite
    - c. Nontronite
    - d. Magnesium bentonite

    composition $Al_2O_3 . 4\ SiO_2 . XH_2O$.
3. The hydrous micas. The general formula is

$$2K_2O.3R'O.8R''_2O_3.24Si_2O.12H_2O$$

where R' denotes a divalent metal, magnesium for example, and R'' denotes a trivalent metal such as aluminum.

X-ray studies of minerals of the groups listed have shown that their constituent molecules, silica and alumina, are arranged in layers. These layers constitute the crystal lattice of a mineral.

In the kaolin group, the layers of silica and alumina alternate, forming a lattice with a molecular ratio of 2 silica to 1 alumina. Owing to replacement of silicon by aluminum within the lattice, ratios of less than 2 to 1 have been observed. Iron may replace aluminum in the lattice without affecting the molecular ratio.

The montmorillonite group is characterized by a lattice consisting of a layer of alumina between two layers of silica. Here the molecular ratio is 4 silica to 1 alumina. Replacement of silicon by aluminum may reduce the ratio to 3 to 1 in certain members of the group, without altering the lattice structure. Iron and magnesium may be present, substituted for the aluminum.

The term "hydrous micas" is a tentative one which serves to designate a group of related minerals having a theoretical formula similar to that of mica. However, these minerals contain less potassium and more water than true micas.

In addition to the clay minerals, certain accessory minerals are to be found in soils. Iron is present in the form of hematite, Fe_2O_3, and goethite, $FeO(OH.)$ Titanium has been found to be present in the form of the mineral, leucoxene, and as the oxide, rutile, TiO_2. Aluminum may be present in the form of diaspore or bohmite, $AlO(OH)$, and gibbsite, $Al(OH)_3$. Silica, SiO_2, may be present in the form of quartz or it may be present in an amorphous state.

Silica-sesquioxide ratio is defined as the molecular ratio of silica (SiO_2) to the combined oxides of iron and aluminum ($Fe_2O_3+Al_2O_3$). Silica-alumina ratio is the molecular ratio of silica to alumina (Al_2O_3). In this study both ratios were found to have a similar relation to physical properties. Therefore, the silica-alumina ratio is not discussed but is included in the paper as a matter of record. Silica-sesquioxide and silica-alumina ratios were determined in the colloidal clay fraction of the soil. This portion of the soil is considered the most active chemically. Usually it is highly weathered and consists mainly of one or more clay minerals of definite chemical composition. Silica-sesquioxide and silica-alumina ratios in the colloidal clay fraction of soils depend largely on the kinds and proportions of clay minerals present.

The colloids are extracted from the soils by means of a supercentrifuge. One hundred grams of soil passing the No. 10 sieve (or 50 grams of clay) are stirred to a smooth slurry with distilled water. The slurry is placed in a disperser and 1,000 milliliters of distilled water containing 1 milliliter of 5N ammonia are added. After dispersing for 2 minutes, the mixture is poured on a No. 270 sieve. The material retained on the sieve is dispersed in the same manner as before and the process continued until the sand remaining on the sieve is washed free of all colloid coating. The combined suspensions containing the soil fraction passing the No. 270 sieve are passed through a supercentrifuge. The centrifuge speed and rate of feed are so adjusted [3] that all particles coarser than 0.001 millimeter are thrown out of suspension inside the centrifuge bowl. The suspension passing through the centrifuge is concentrated with fine-grade Pasteur-Chamberland filter candles using suction. The material thrown out of suspension is removed from the bowl and redispersed in the filtrate from the candles. The resulting suspension is again centrifuged. The process is repeated until the centrifugate is practically clear, showing that all of the dispersible colloid has been removed.

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[1] Standard Specifications for Materials for Stabilized Base Course, M 56-38 and Standard Specifications for Stabilized Surface Course, M 61-38, American Association of State Highway Officials.

[2] Minerals Present in Soil Colloids, by S. B. Hendricks and L. T. Alexander. I. Descriptions and Methods of Identification. Soil Science, vol. 48, No. 3, 1939, p. 258.

The colloids are concentrated to a slurry by filtration with the candles. The slurry is dried at 105° C., powdered, and the powder stored in vials for analysis. The analyses are made by standard procedures for soil analysis.[4]

Base or ion exchange is defined as the substitution of a base for another base, or for hydrogen, in the soil. This action can best be illustrated by the description of a laboratory experiment. A sample of soil that showed an acid reaction was placed in a funnel and neutral potassium chloride was leached through the soil. The leachate was tested and found to be acid; on analysis the soil was found to contain more potassium than the original soil. The reaction can be written: Hydrogen clay+potassium chloride⇆potassium clay+hydrochloric acid. The process is reversible and follows well-defined chemical laws.

Base exchange determinations involve measuring (1) the base exchange capacity and (2) the kind and amount of the exchangeable bases. The method may be outlined as follows:

The portion of the soil sample passing the No. 40 sieve is leached with neutral ammonium-acetate solution. The leachate contains all the exchanged bases and the soil retains the absorbed ammonia which displaced the bases. The leachate is then analyzed for the amount and kind of bases exchanged. The soil is washed to remove the excess ammonium acetate and is then analyzed for the absorbed ammonia content. From the amount of ammonia absorbed, the base exchange capacity of the soil is calculated. The sum of the exchanged bases and the base exchange capacity are recorded in milliequivalents per 100 grams of sample taken from the roadway.

SILICA-SESQUIOXIDE RATIO INVESTIGATED

A study of the significance of the silica-sesquioxide ratio was made on 19 samples of soil-aggregate base courses taken from roads in service in Alabama and Georgia. The results of the physical tests on these samples and the condition of the road surfaces at the time the samples were taken are shown in table 1. Chemical analyses were made of the colloid fractions of these samples and the results are given in table 2, including the silica-sesquioxide ratios and the silica-alumina ratios.

These base courses had been surface treated with single or double applications of bituminous material and mineral aggregate. Since the resulting surfaces had a total thickness of only 1 inch or less, the condition of the surface was assumed to be directly dependent on the condition of the base course.

The Standard Specifications for Materials for Stabilized Base Course, M 56–38, of the A. A. S. H. O., require that the dust ratio, *B/A* (ratio of the fraction passing the No. 200 sieve to the fraction passing the No. 40 sieve) shall not exceed 0.50, and that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 6.

Samples 2, 5, 11, 14, and 19 (table 1) met the requirements for dust ratio, liquid limit, and plasticity index of the A. A. S. H. O. specifications for base courses; and it is noted also, in these cases, that the roads were in satisfactory condition. Samples 1, 3, and 10 had higher plasticity indexes than the specifications permit but conformed in all other details. The roads from which these samples were taken were satisfactory.

TABLE 1.—*Physical test data of Alabama and Georgia base-course materials as related to road conditions*

Sample No.	Sieve analysis of samples, percentage passing—								Ratio *B/A*	Hydrometer analysis, percentage of total sample		Physical characteristics of material passing the No. 40 sieve						
														Shrinkage		Moisture equivalent		
	1½-inch sieve	1-inch sieve	¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve (*A*)	No. 200 sieve (*B*)		Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Liquid limit	Plasticity index	Limit (S)	Ratio (R)	Centrifuge	Field (FME)	Condition of road surface
1	--------	100	99	90	82	78	62	23	0.37	6	15	25	9	16	1.8	13	20	Satisfactory.
2	--------	--------	100	90	64	56	53	18	.34	4	11	18	5	13	1.9	9	14	Do.
3	--------	--------	100	98	96	83	48	19	.40	2	16	25	9	17	1.8	14	19	Do.
4	100	98	93	88	84	72	47	23	.49	3	19	42	16	18	1.7	20	23	Do.
5	--------	100	99	98	96	93	65	33	.51	13	18	19	4	13	2.0	10	15	Do.
6	93	90	88	85	81	74	50	31	.62	13	17	27	11	14	1.9	21	19	Fair.
7	--------	--------	100	99	97	94	70	45	.64	21	21	25	8	14	1.8	18	20	Do.
8	97	89	82	71	63	54	36	21	.58	11	9	44	21	18	1.7	23	32	Do.
9	94	88	78	61	53	46	32	19	.59	7	8	28	10	14	1.8	16	20	Do.
10	100	99	98	89	69	57	47	20	.42	5	12	24	10	14	1.8	13	18	Satisfactory.
11	--------	100	99	86	67	57	47	22	.47	7	10	18	4	14	1.9	12	14	Do.
12	--------	--------	--------	--------	--------	100	89	17	.19	2	15	32	10	23	1.5	12	30	Poor.
13	--------	--------	--------	--------	--------	100	94	21	.22	1	20	36	14	22	1.6	15	38	Do.
14	--------	--------	--------	--------	--------	100	74	20	.27	3	16	25	6	19	1.7	10	19	Satisfactory.
15	--------	--------	--------	--------	--------	100	59	21	.36	3	16	28	8	18	1.7	11	19	Do.
16	--------	--------	--------	--------	--------	100	61	20	.32	2	17	33	10	20	1.7	12	24	Do.
17	100	99	95	73	56	48	39	17	.44	5	12	28	12	16	1.8	18	20	Do.
18	--------	100	97	77	59	50	35	14	.40	4	11	31	11	16	1.8	14	24	Do
19	--------	100	95	80	65	58	45	15	.33	4	10	25	4	16	1.7	13	20	Do.

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[3] The Fractionation, Composition, and Hypothetical Constitution of Certain Colloids Derived from the Great Soil Groups, by I. C. Brown and H. G. Byers, U. S. Department of Agriculture, Technical Bulletin No. 319, p. 8, 1932.

[4] Method and Procedure of Soil Analysis Used in the Division of Soil Chemistry and Physics, by W. O. Robinson, U. S. Department of Agriculture, Circular No. 139, 1939.

Samples 4, 12, 13, 15, 16, 17, and 18 had higher plasticity indexes and liquid limits than the A. A. S. H. O. specifications permit but the dust ratios were satisfactory. Of the seven roads of which these seven samples were representative, five, Nos. 4, 15, 16, 17, and 18, were in a satisfactory condition, and two, Nos. 12 and 13, were in poor condition. The remaining four samples, Nos. 6, 7, 8, and 9, had excessive dust ratios (*B*/*A* values, table 1), and high plasticity indexes and liquid limits. The road conditions in these four instances were reported as fair.

TABLE 2.—*Chemical analyses of colloids from Alabama and Georgia base courses*

Sample No.	SiO_2	Fe_2O_3	TiO_2	Al_2O_3	Loss on ignition	Molecular ratios	
						$\frac{SiO_2}{Al_2O_3+Fe_2O_3}$	$\frac{SiO_2}{Al_2O_3}$
	Percent	*Percent*	*Percent*	*Percent*	*Percent*		
2	35.0	8.3	4.4	35.2	17.4	1.5	1.7
5	31.7	12.7	2.4	35.8	16.7	1.2	1.5
11	32.3	11.7	3.9	35.7	17.7	1.3	1.5
14	30.1	25.7	2.7	28.3	13.5	1.1	1.8
19	42.2	11.4	1.8	31.5	12.4	1.9	2.3
1	37.7	12.5	4.6	31.7	14.5	1.6	2.0
3	34.8	14.2	3.4	32.3	14.6	1.4	1.8
10	33.6	10.9	4.9	34.5	17.1	1.4	1.7
4	34.9	15.2	4.1	32.2	14.4	1.4	1.8
12	42.9	10.3	2.8	30.0	12.8	2.0	2.4
13	42.6	10.3	2.7	31.2	12.8	1.9	2.3
15	35.6	17.7	2.1	30.3	13.3	1.5	2.0
16	35.6	19.7	2.9	28.6	12.8	1.5	2.1
17	39.3	15.4	2.3	29.5	12.4	1.7	2.3
18	38.8	15.8	2.2	29.3	12.5	1.7	2.3
6	39.2	12.0	2.1	31.8	13.9	1.7	2.1
7	41.2	9.9	1.7	32.5	13.7	1.8	2.2
8	41.5	8.4	2.2	34.2	14.6	1.8	2.1
9	42.8	8.2	2.6	33.7	14.6	1.9	2.2

The silica-sesquioxide ratio of soil colloids is controlled by the kind and amount of clay minerals present. The relation of the silica-sesquioxide ratio to the character and behavior of soils is hence a resultant of effect of the clay minerals. Soils with colloids having ratios higher than 2, indicating the presence of montmorillonite group clay minerals, tend to have greater volume changes than soils with ratios less than 2.

Values for the volume change and lineal shrinkage of the 19 base-course materials from Georgia and Alabama are given in table 3. Chemical analyses and silica-sesquioxide and silica-alumina ratios of six additional soils are given in table 4. Values for volume change and lineal shrinkage of these soils are given in table 5. Samples 38, 39, and 40 represent subgrade soils from Alabama and Georgia and samples 41, 42, and 43 represent soils from the vicinity of Washington, D. C. It will be noted that the silica-sesquioxide and silica-alumina ratios of the three northern soils are greater than those for the three southern soils.

In figure 1 are shown the relations between the lineal shrinkage and the silica-sesquioxide (SiO_2/R_2O_3)* and silica-alumina (SiO_2/Al_2O_3) ratios for samples 1 to 19 inclusive (tables 2 and 3) and samples 38 to 43 inclusive (tables 4 and 5). The curves of figure 1 are straight lines plotted by the method of least squares and show a definite trend toward an increase in lineal shrinkage with an increase in either the silica-sesquioxide or silica-alumina ratio. Physical test constants other than lineal shrinkage do not show this trend.

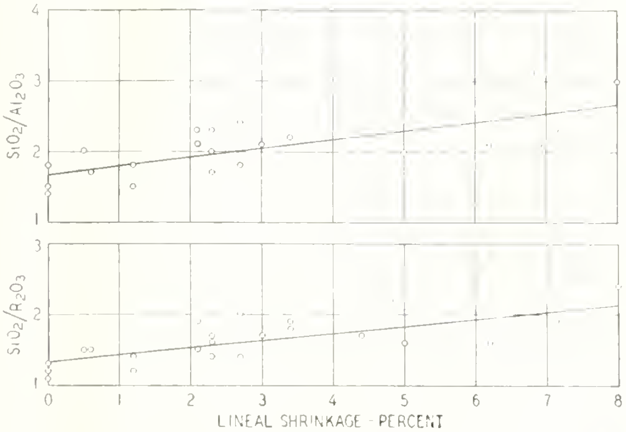

FIGURE 1.—THE RELATION BETWEEN SiO_2/R_2O_3, SiO_2/Al_2O_3 RATIOS AND LINEAL SHRINKAGE OF SOILS.

TABLE 3.—*Volume change and lineal shrinkage of Georgia and Alabama base-course materials*

Sample No.	Volume change $C_f = R\,(FME-S)$	Lineal shrinkage $100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$
2	1.9	0.6
5	4.0	1.2
11	0	0
14	0	0
19	6.8	2.1
1	7.2	2.3
3	3.6	1.2
10	7.2	2.3
4	8.5	2.7
12	8.5	2.7
13	25.6	7.2
15	1.7	.5
16	6.8	2.1
17	7.2	2.3
18	14.4	4.4
6	9.5	3.0
7	10.8	3.4
8	23.8	6.9
9	10.8	3.4

TABLE 4.—*Chemical analyses of colloid fraction of subgrade soils*

Sample No.	SiO_2	Fe_2O_3	TiO_2	Al_2O_3	Loss on ignition	Molecular ratios	
						$\frac{SiO_2}{Al_2O_3+Fe_2O_3}$	$\frac{SiO_2}{Al_2O_3}$
	Percent	*Percent*	*Percent*	*Percent*	*Percent*		
38	30.8	11.7	2.9	36.8	17.1	1.2	1.4
39	37.4	14.1	4.2	31.7	13.3	1.6	2.0
40	38.4	14.6	2.8	31.8	12.7	1.6	2.1
41	46.0	5.2	3.6	31.7	12.4	2.2	2.5
42	47.0	12.7	1.2	25.8	11.2	2.4	3.1
43	48.5	9.2	1.0	28.0	9.0	2.4	3.0

TABLE 5.—*Volume change and lineal shrinkage of subgrade soils*

Sample No.	Volume change $C_f = R\,(FME-S)$	Lineal shrinkage $100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$
38	0	0
39	16.5	5.0
40	21.0	6.2
41	16.2	4.8
42	23.4	6.8
43	28.5	8.0

Since there is a relationship between the silica-sesquioxide ratios and the physical properties of soils, there should be in turn a relationship between the service behavior and silica-sesquioxide ratio. That this is true is shown in table 6. Five base courses with soil aggregates conforming to A. A. S. H. O. specifications contained colloids with silica-sesquioxide ratios from 1.1 to 1.9. All of the roads represented by these samples gave satisfactory performances. Of the remaining 14 soil-aggregates which conformed only in part with the A. A. S. H. O. specifications, those with silica-sesquioxide ratios below 1.7 represent satisfactory roads, those having ratios of 1.7 or 1.8 are borderline cases varying between satisfactory and fair, and those with ratios of 1.9 and 2.0 are unsatisfactory.

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*$R_2O_3 = Al_2O_3 + Fe_2O_3$.

TABLE 6.—*Comparison of condition of roads with chemical characteristics of colloids from base course materials from Alabama and Georgia*

Sample No.	Type of material	Condition of road	Silica-sesquioxide ratio
SOILS CONFORMING WITH A. A. S. H. O. SPECIFICATIONS (EXCEPTIONS NOTED)			
[1] 2	Georgia pebble soil	Satisfactory	1.5
5	Topsoil	do	1.2
11	Clay gravel	do	1.3
14	Sand clay	do	1.1
[1] 19	Clay gravel	do	1.9
SOILS WITH EXCESSIVE PLASTICITY INDEX			
1	Georgia pebble soil	Satisfactory	1.6
3	Sand clay	do	1.4
10	Clay gravel	do	1.4
SOILS WITH EXCESSIVE LIQUID LIMIT AND PLASTICITY INDEX			
4	Sand clay	Satisfactory	1.4
[1] 12	do	Poor	2.0
[1] 13	do	do	1.9
[1] 15	do	Satisfactory	1.5
[1] 16	do	do	1.5
17	Clay gravel	do	1.7
18	do	do	1.7
SOILS WITH EXCESSIVE DUST RATIO, LIQUID LIMIT, AND PLASTICITY INDEX			
6	Topsoil	Fair	1.7
7	do	do	1.8
8	Clay gravel	do	1.8
9	do	do	1.9

[1] Does not conform to A. A. S. H. O. gradation specification.

The base courses were from 5 to 8 inches thick so that it was not considered likely that the subgrade had a marked effect on the surface behavior of the roads represented by samples 1 to 19, tables 1, 2, and 3.

BASE EXCHANGE RELATED TO SURFACE COURSE PERFORMANCE

Since a relationship between the results of chemical and physical tests and the service behavior of soil-aggregates in base courses was shown, there should be a relationship between chemical factors and the behavior of soil-aggregates in surface courses. To test this assumption, a study of base-exchange characteristics was made on soil-aggregate samples from the surface courses of 14 roads in Maryland and 4 roads in Delaware.

Base exchange was considered to be of most significance in these surface courses because a large percentage of the surfaces tested had been treated with sodium chloride or calcium chloride. Base exchange measurements take into account the effect of the addition of these chemicals, whereas silica-sesquioxide measurements are independent of the addition of water-soluble chemicals.

The condition of these 18 soil-aggregate surface courses and the results of the physical tests on samples taken from them are given in table 7. Samples 20 to 23, inclusive, were taken from 4 road surfaces in Delaware that subsequently were to serve as base courses for bituminous surfaces. However, at the time they were sampled and their condition observed, they were being used as surface courses and therefore they are considered as such in this study.

The Standard Specifications for Materials for Stabilized Surface Course, M 61–38, of the A. A. S. H. O., require that the dust ratio, B/A, shall not exceed 2/3, and that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 35 and a plasticity index not less than 4 and not greater than 9.

Samples 20 to 23, inclusive, failed to meet the specification requirements for surface courses in that their plasticity indexes were too low. However, the road conditions were satisfactory in the sections represented by samples 20, 21, and 22. The road surface represented by sample 23 was unconsolidated and very dusty in summer.

Materials represented by samples 24 to 37, inclusive, table 7, were taken from surface courses of Maryland roads. Sample 37 met the requirements of the A. A. S. H. O. specifications for surface courses, except for grading, but the road surface was soft and rutted under light loads. The plasticity indexes of samples 24, 30, 33, 34, 35, and 36, were low. The roads from which samples 24, 33, and 34 were taken were in good condition. The road represented by sample 30 became soft when wet and the surfaces of the sections represented by samples 35 and 36 were corrugated due to a deficiency of binder.

The section from which sample 25 was taken was in good condition and that represented by sample 26 was in faily good condition, although the dust ratios of these samples were higher and the plasticity indexes lower than the specifications permit. Sample 32, which had an excessively high plasticity index, was representative of a road having a number of pot holes (due in part perhaps to lack of maintenance). Sample 31, which had a high dust ratio, was taken from a road that was badly rutted and soft when wet. The remaining three samples, Nos. 27, 28, and 29, which had higher dust ratios, plasticity indexes, and liquid limits than the A. A. S. H. O. specifications permit, were taken from roads that were badly rutted.

Of the Delaware roads, the two represented by samples 20 and 21 gave the best service.[5] The third (No. 22) was satisfactory but not quite as good as the first two. The fourth sample (No. 23) represented material that was unsatisfactory. It did not consolidate and was very dusty in dry weather.

Base exchange data for the Delaware and Maryland surface courses are given in table 8. They show that, of the four samples from Delaware, sample 23 had a much lower base exchange capacity than the other three. These results indicate that the soil mortar of sample 23 contained a much less effective binder than the other three soils which show highly weathered and effective binders. Base exchange data and condition of the Delaware roads are compared in table 9.

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[5] Reported by Maxwell P. Harrington, Soils Engineer, Delaware State Highway Department by letter—"When placed in the roadway they consolidate rapidly and become exceedingly hard. They are impervious to water when consolidated and do not become even slightly soft or muddy."

TABLE 7.—*Physical test data of Delaware and Maryland surface course material as related to road conditions*

Sample No.	Sieve analysis of samples, percentage passing— 1½-inch sieve	1-inch sieve	¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve (A)	No. 200 sieve (B)	Ratio B/A	Hydrometer analysis: Silt, 0.05 to 0.005 millimeter	Clay, smaller than 0.005 millimeter	Physical characteristics of material passing the No. 40 sieve: Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio	Moisture equivalent: Centrifuge	Moisture equivalent: Field	Condition of road surface
20	------	------	100	98	94	94	55	13	0.24	5	8	21	1	23	1.6	14	22	Smooth and well compacted.
21	100	97	87	82	75	75	52	11	.21	3	8	19	0	21	1.7	13	21	Do.
22	------	------	100	99	98	98	68	13	.19	2	11	16	0	21	1.7	7	28	Do.
23	------	------	------	------		100	62	7	.11	3	4	15	0	26	1.6	3	22	Unconsolidated. Dusty in summer
24	------	------	100	------	96	74	40	21	.53	10	9	18	2	------	------	13	------	Smooth and well compacted.
25	------	87	79	------	59	44	31	23	.74	13	9	25	3	------	------	19	------	Surface tight, but rough due to lack of maintenance.
26	------	------	100	------	88	62	40	28	.70	19	6	20	3	------	------	13		Surface has loose mulch on top and a few soft spots.
27	------	94	88	------	69	54	43	31	.72	16	13	41	10	------	------	34	------	Surface badly cut up and rutted.
28	------	97	90	------	63	52	39	31	.80	12	17	39	11	------	------	31	------	Rutted badly, soft when wet.
29	------	100	99	------	92	84	71	62	.87	34	25	39	10	------	------	32	------	Do.
30	------	------	100	------	69	43	23	13	.57	7	5	22	3	------	------	14	------	Surface smooth, soft when wet.
31	------	91	90	------	81	73	59	42	.71	21	17	34	8	------	------	26	------	Rutted, soft when wet.
32	------	95	94	------	62	48	35	16	.46	4	11	31	11	------	------	18	------	Pot holes, may be due in part to lack of maintenance.
33	------	98	95	------	67	54	29	12	.41	5	7	19	3	------	------	12	------	Satisfactory. Loose mulch on top.
34	------	100	99	------	82	66	43	12	.28	4	7	16	0	------	------	6	------	Smooth in good condition.
35	------	97	89	------	57	47	30	13	.43	2	6	17	0	------	------	8	------	Surface corduroyed. Not enough binder.
36	------	94	84	------	55	46	29	7	.24	2	5	15	0	------	------	5	------	Do.
37	------	100	99	------	88	77	55	28	.51	16	9	22	4	------	------	14	------	Surface soft, rutted.

TABLE 8.—*Base exchange data obtained with Delaware and Maryland soils*

Sample No.	Milli-equivalents[1] of base exchange capacity: Per 100 grams soil passing No. 40 sieve	Per 100 grams total sample	Predominant exchangeable base	pH
20	4.0	2.0	Ca and Mg	4.8
21	2.9	1.1	Ca	4.8
22	1.9	1.3	Ca	4.4
23	1.2	.7	Ca	5.0
24	3.3	1.3	Ca	7.6
25	6.7	2.1	Ca	7.8
26	2.3	.9	Ca	7.4
27	8.0	3.4	Ca	7.4
28	6.8	2.7	Ca	4.7
29	7.5	5.3	Ca	5.8
30	10.0	2.3	Ca	7.7
31	5.3	3.1	Ca	5.5
32	6.9	2.4	Ca	4.8
33	3.6	1.0	Ca	4.1
34	2.9	1.2	Ca	5.1
35	3.8	1.1	Ca	4.8
36	2.0	.6	Na and Ca	6.3
37	5.0	2.8	Ca	7.7

[1] $\text{Milli-equivalent} = 1000 \times \frac{\text{weight of substance in grams}}{\text{chemical equivalent weight of the substance}}$

TABLE 9.—*Comparisons of road conditions with base exchange data of Delaware soils*

Sample No.	Type of material	Condition	Base exchange capacity milli-equivalents per 100 grams total sample
20	Clay gravel	Smooth and well compacted	2.0
21	do	do	1.1
22	do	do	1.3
23	do	Unconsolidated. Dusty in summer	.7

According to Hogentogler and Willis[6] the stability of a soil depends on the cohesiveness of the binder and the strength and permanency of the adhesion that can be developed between the coarser soil particles and binders. Therefore, if base exchange measurements indicate the effectiveness of the binder, the effectiveness in turn should also be indicated by physical tests if the diluting action of the sand can be eliminated. To test this assumption, the samples from Delaware roads were sieved through the No. 200 sieve and the physical tests made on the minus No. 200 material.

The test results, table 10, show that both the liquid limit and the colloid content of sample 23A are about half those of the other three. The low colloid content and low liquid limit are indicative of the low efficiency of the binder in the case of this soil-aggregate. Higher colloid contents in the other samples produced higher physical test constants and a more cohesive binder.

TABLE 10.—*Mechanical analysis and physical characteristics of Delaware soils passing the No. 200 sieve*

MECHANICAL ANALYSIS

Sample No.	Particles smaller than 2 millimeters (percentage by weight): Coarse sand, 2.0 to 0.25 millimeter	Fine sand, 0.25 to 0.05 millimeter	Silt, 0.05 to 0.005 millimeter	Clay, smaller than 0.005 millimeter	Colloids, smaller than 0.001 millimeter
20A	0	10	59	31	21
21A	0	5	49	46	28
22A	0	14	45	41	18
23A	0	12	39	49	11

PHYSICAL CHARACTERISTICS OF MATERIAL PASSING NO. 200 SIEVE

Sample No.	Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio
20A	45	10	22	1.6
21A	44	14	20	1.7
22A	42	10	20	1.7
23A	21	0	20	1.7

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[6] Stabilized Soil Roads, by C. A. Hogentogler and E. A. Willis, PUBLIC ROADS, vol. 17, No. 3, May 1936, p. 45.

For the 14 Maryland roads, samples 24 to 37, the condition of the roads and the base exchange characteristics of the corresponding soils are listed in table 11.

TABLE 11.—*Base exchange data and road conditions for Maryland soils conforming and not conforming with A. A. S. H. O. specifications*

Sample No.	Type of material	Conformity with specifications	Milliequivalents base exchange capacity per 100 grams total sample	Condition of road
[1] 37	Rock-soil	Conforms except for grading.	2.8	Surface soft, rutted.
24	Limestone screenings.	Plasticity index too low.	1.3	Smooth, well compacted.
30	do	do	2.3	Smooth, soft when wet.
33	Sand-gravel	do	1.0	Satisfactory, loose mulch on top.
34	Sand-clay-gravel	do	1.2	Smooth, in good condition.
35	Sand-gravel	do	1.1	Surface corduroyed, not enough binder.
36	do	do	[2] .6	Do.
[1] 25	Topsoil and limestone.	Plasticity index too low and dust ratio too high.	2.1	Surface tight.
[1] 26	Limestone screenings.	do	.9	Surface has loose mulch on top and soft spots.
32	Sand-gravel	Plasticity index too high.	2.4	Pot holes, may be due in part to lack of maintenance.
[1] 27	Gravelly	Liquid limit, plasticity index and dust ratio, all too high.	3.4	Surface cut up and rutted
[1] 28	Stone fragments	do	2.7	Rutted, soft when wet.
[1] 29	do	do	5.3	Do.
[1] 31	do	Dust ratio too high	3.1	Do.

[1] Does not conform to grading specifications.

[2] Predominant exchangeable base in this soil is sodium. In all other soils calcium predominates.

Comparing the requirements of the A. A. S. H. O. specifications for surface courses with the physical test results and the conditions of the road surfaces, it may be noted that samples failing to meet the A. A. S. H. O. specifications due to a low plasticity index represented roads in both good and unsatisfactory condition. However, the base exchange capacity of the soil, calculated on the basis of 100 grams total sample, shows a striking correspondence with the service behavior of the road. Six roads (represented by samples 37, 32, 27, 28, 29, and 31) with soils of base exchange capacities ranging from 2.4 to 5.3 were in an unsatisfactory condition with surfaces badly cut up and rutted. The surface represented by sample 30, with a base exchange capacity of 2.3, was in good condition in dry weather but became soft in wet weather. Four samples (Nos. 33, 35, 36, and 26) having base exchange capacities of from 0.6 to 1.1 were from roads that were corduroyed or had loose mulch on top. The remaining three samples were all from roads in good condition, and for these samples the base exchange capacity varied from 1.2 to 2.1. Hence it is possible to correlate the service behavior of these roads with the base exchange capacity of the soil-aggregate in the road. The soils can be arranged into three groups according to their base exchange capacities as follows:

1. High base exchange capacity. The binder is overactive, causing detrimental effects resulting in roads with poor surfaces.
2. Medium base exchange capacity. This gives the desirable results of consolidation and bonding of the road surface.
3. Low base exchange capacity. The resulting detrimental effects are loose and unconsolidated material.

Soils can be classified in this manner even if they show uniform physical test constants because the chemical characteristics of soils seem to be the fundamental factors controlling physical properties. Particle size thus becomes a secondary factor. In some cases its effect is greatly modified by chemical variations.

Soils conforming to A. A. S. H. O. specifications are, in general, satisfactory. Those not conforming to the specifications may be satisfactory road materials because of their particular chemical properties. These chemical properties appear to be more important than has been generally realized.

Table 8 contains a column showing the results of hydrogen ion (pH) measurements on the Delaware and Maryland soils. They are included as a matter of record, but pH, in this study, has not been found to have any relation to either physical tests or service behavior.

SUMMARY

The data presented warrant the following conclusions:

1. Not all soils can be adequately classified for road-building purposes by physical tests alone. The chemistry of the soils must also be considered in many cases.
2. There is a general relationship between the silica-sesquioxide ratios of the colloid fractions of soils and the lineal shrinkages of the same soils.
3. The condition survey showed that roads having soil-aggregate base courses whose colloid fractions had low silica-sesquioxide ratios were in better condition than those with higher ratios.
4. Base exchange measurements indicate the activity of the binder portion of the soil.
5. The base exchange capacity of the binder soil was directly related to the service behavior of 18 road surfaces examined. Surface courses having base exchange capacities above 2.2 milliequivalents per 100 grams of sample were soft and rutted. With the exception of sample 21, those having base exchange capacities below 1.2 milliequivalents were corduroyed and did not consolidate. For values of the milliequivalent from 1.2 to 2.1, inclusive, the surfaces of the roads were in satisfactory condition.

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For the condition of ultimate failure in the stabilometer test, the value of C is taken as zero, and with reference to the computed values of C, table 2, it is seen that only for the condition of 100 percent maximum density is this soil able to deform in place without failure under a principal stress difference of 6,000 pounds per square foot which corresponds to a unit surface load of 10,344 pounds per square foot. For this particular table, a decrease of 4.8 percent in moisture content (24.5 to 19.7 percent) is attended by a relatively small gain in dry density (from 103 to 105 pounds per cubic foot). However, the increase in C corresponding to this moisture difference is quite large and is indicative of benefits derivable from having a well-compacted and stable subgrade.

TABLE 2.—*The effect of moisture content and compaction on the modulus, C, of a clay soil and on the settlement, S_L, of this soil when under a uniformly loaded circular area of radius 6.5 inches, $\mu=\frac{1}{2}$*

Soil moisture content	Dry density		Modulus, C, corresponding to principal stress difference, $v-l$				Settlement, S_L, corresponding to unit load, $p=\frac{v-l}{0.58}$, $\frac{z}{a}=0$			
			$v-l=500$	$v-l=1{,}000$	$v-l=3{,}000$	$v-l=6{,}000$	$p=862$	$p=1{,}724$	$p=5{,}172$	$p=10{,}344$
Percentage of dry weight	*Pounds per cubic foot*	*Percentage of maximum density*	*Pounds per square foot*	*Pounds per square foot*	*Pounds per square foot*	*Pounds per square foot*	*Inches*	*Inches*	*Inches*	*Inches*
35.2	86	82	50,000	[1] 0	[1] 0	[1] 0	0.17	(2)	(2)	(2)
30.5	92	88	170,000	80,000	[1] 0	[1] 0	.05	0.21	(2)	(2)
24.5	103	98	600,000	440,000	120,000	[1] 0	.01	.04	0.42	(2)
19.7	105	100	900,000	790,000	590,000	250,000	.01	.02	.09	0.40

[1] Samples failed at these principal stress differences.
[2] Failure.

CONCLUSIONS

On the basis of the earlier report [7] and on the computations and data in this report, it is believed that the following general conclusions are warranted.

1. For a uniform load on a circular area at the earth surface, failure of a cohesive supporting earth is most likely to begin at any point of a basin-shaped surface intersecting the axis of symmetry at a depth equal to about 0.7 of the radius of the loaded circular area and extending to the perimeter. For points on this surface, the principal stress differences have maximum values.

2. Since stresses of a certain magnitude may be insufficient to cause failure of the subgrade and yet sufficient to cause considerable settlement, depending on the properties of the subgrade, a knowledge of the stress-deformation characteristics of the subgrade soil is absolutely necessary.

3. It is believed that development of a method of estimating deflections of the subgrade on the basis of laboratory test data and without the necessity of loading tests in the field is a desirable objective.

4. A method is described for determining the modulus of deformation of cohesive soils by means of the triaxial compression device. In general this is a secant modulus that diminishes in magnitude as the principal stress difference is increased.

5. By substituting the appropriate value for this modulus in integrated expressions similar to those that apply to elastic behavior, it is possible to make some sort of estimate of the deflection of the supporting soil under load.

6. It is shown that the movement under stress diminishes according to determinable relations as the thickness of pavement is increased, assuming that both lateral yield and compaction within the flexible pavement itself is relatively small.

7. Values of settlement computed from theory and stabilometer test data indicate the possibility of very materially reducing deflections under wheel loads by having the subgrade compacted to maximum density.

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[7] Stresses Under Circular Loaded Areas, by L. A. Palmer. Proceedings of the Highway Research Board, vol. 19, 1939.

(Continued from page 192)

It would appear from the speed distributions that the effect of highway lighting at this hazardous location was a reduction in the average speed when the pavement was wet. However, concrete pavement has a tendency to appear slippery at night when wet, whether the light is from an overhead source or from the headlights of vehicles. Such accentuation of the appearance of slipperiness may account for this marked reduction in speed, but it is impossible from the results obtained to determine to what extent, if any, the overhead lights influenced vehicle speeds when the pavement was wet.

The speed counter was operated continuously for 96 hours at a position on the tangent at the top of the hill. This counter merely classified the vehicles by their speeds into 10 groups, and the total number of vehicles in each group was manually recorded at the end of each hour. Despite the fact that speeds at night under lights at this position may not be representative of normal behavior, this phase of the study is of particular interest because it represents perhaps the longest continuous record of vehicle speeds ever collected.

Table 4 shows the average speed of vehicles and the distribution of speeds in 10 groups. As might be expected, the average speed during daylight, 38.5 miles per hour, is faster than the average speed at night. The average speed with the lights on, 35.0 miles per

TABLE 4.—*Percentage of vehicles traveling in various speed groups during daytime and nighttime while pavement was dry and while pavement was wet* [1]

Speed group, miles per hour	Daylight hours, 7 a. m. to 5 p. m.		Night hours, 6 p. m. to 6 a. m.		
			Pavement dry		Pavement wet, lights on
	Dry pavement	Wet pavement	Lights off	Lights on	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Below 19.1	2.8	3.6	2.6	5.2	2.4
19.1–25.0	3.2	5.6	4.6	6.4	4.5
25.1–29.7	7.5	10.0	9.8	11.1	11.8
29.8–35.1	19.1	22.4	24.6	25.4	29.5
35.2–41.4	30.1	32.7	31.4	25.9	31.9
41.5–45.4	17.0	13.6	14.0	13.6	11.5
45.5–50.3	12.6	8.0	7.6	8.4	5.8
50.4–56.3	5.7	3.1	3.4	2.8	2.1
56.4–59.5	1.1	.5	.9	.5	.2
Over 59.5	.9	.5	1.1	.7	.3
Average speed	38.5	36.5	36.8	35.0	35.8
Vehicles per hour	197	174	84	82	73
Number of vehicles studied	5,886	1,722	1,989	966	874

[1] Data for 8 hours of study between 6 and 7 a. m. and between 5 and 6 p. m. and for 1 hour at night with pavement wet, lights off, are not included in this table

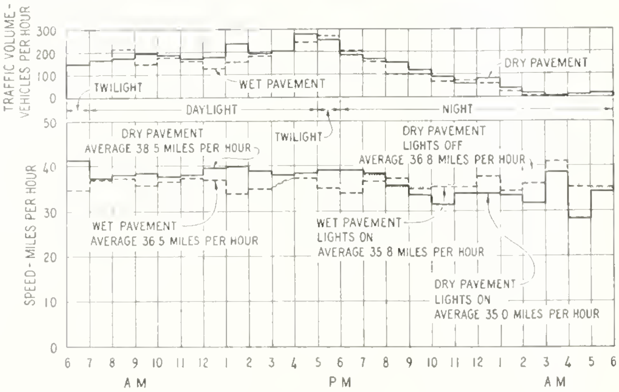

Figure 12.—Traffic Volumes and Average Speeds by Hours.

hour, is less than the average of 36.8 miles per hour found with the lights off. Wet pavement resulted in a decrease of speed in daytime, but a slight increase at night with the highway illuminated, a change that is inexplicable. It is significant that where a reduction of average speed occurs (table 4) it results from a general lowering of speeds in all speed ranges rather than because of a marked decrease in the number of vehicles in the higher speed groups. Under all conditions, some vehicles moved at 60 miles per hour or faster despite the 35-mile speed limit.

Figure 12 shows graphically the variation in average speed and traffic volume by hours. The consistency of the records is shown very clearly in this figure, for in no daylight hour was the average speed on wet pavement as great as that on dry pavement. This condition was reversed when the highway was lighted, as in the early evening, and thereafter throughout the night the average speed on wet pavement remained consistently higher than that when the pavement was dry.

SUMMARY

In interpreting the significance of the results of this investigation, several factors should be borne in mind.

1. The traffic volumes were relatively low, and the effect of lights on the speed of traffic as shown here may be entirely different from that for greater traffic volumes.

2. The alinement and grade of the short length of road studied may have prevented drivers from driving normally under any one condition.

3. The novelty of the lights being off for the first time in several years may well have had an influence on drivers accustomed to using the road.

4. The number of vehicles for which the speeds were recorded at night was not great.

5. The final criterion of effectiveness of lighting installations should be the effect on safety as revealed by before- and after-accident records.

These considerations almost preclude comparison of the results obtained in this study with those obtained elsewhere. Results of the study, however, seem to permit the following conclusions:

1. There are measureable differences in the behavior of drivers during daylight and darkness. These differences are most apparent in the transverse position of vehicles and in the frequency of passing. There is evidence that speed is also affected, but in this study the evidence cannot be considered conclusive.

2. The behavior of drivers operating on a lighted highway conforms very nearly to their behavior in daylight but does not conform to their behavior on unlighted highways at night, insofar as transverse position and passing frequency are concerned.

3. Conditions of illumination, as well as alinement and weather conditions, have no apparent effect on the normal distribution of time spacings between vehicles.

Other findings of general interest are:

4. A posted speed limit, when unenforced, has little effect upon the speeds of vehicles. Speeds as high as 82 miles per hour were recorded on the study section where the posted limit was 35 miles per hour, and hourly averages were seldom below this legal limit. This speed limit appears unreasonably low because the locality is distinctly rural in character although within corporate limits.

5. There is a certain minority of drivers on the road at all times who are prone to take risks. This is brought out rather clearly in the passing study, in which drivers were found passing where sight distances were entirely too short for safety.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,544,939	$ 1,265,304	73.6	$ 5,060,351	$ 2,516,990	196.0	$ 1,362,370	$ 678,110	47.6	$ 1,784,752
Arizona	1,028,980	681,907	47.7	1,308,250	922,445	60.8	423,225	233,341	9.6	777,956
Arkansas	4,047,747	1,890,989	107.8	1,345,803	667,127	56.0	468,153	233,770	18.5	246,174
California	4,689,174	2,439,857	100.2	8,470,058	4,436,259	114.8	2,109,617	1,089,870	50.4	809,415
Colorado	1,983,426	1,091,833	174.5	2,038,323	1,148,091	82.0	491,917	285,670	82.5	1,833,683
Connecticut	844,847	416,697	8.3	2,116,748	1,025,435	18.2	161,683	78,844	1.0	999,062
Delaware	275,496	137,748	12.0	1,917,176	931,150	20.6				1,040,813
Florida	1,253,119	626,559	33.3	2,757,533	1,368,488	83.6	1,548,161	743,451	26.0	1,996,268
Georgia	2,379,521	1,182,024	154.2	7,611,685	3,806,342	293.7	1,929,078	964,539	94.0	4,728,258
Idaho	1,356,626	826,505	146.4	1,148,359	702,959	60.5	104,377	64,380	5.7	1,327,372
Illinois	4,938,623	2,442,204	116.3	7,625,792	3,812,531	154.0	2,580,900	1,125,950	80.7	1,920,753
Indiana	2,668,729	1,331,306	63.4	7,799,859	3,749,773	135.0	2,529,760	1,232,755	38.1	511,267
Iowa	4,250,941	1,953,780	147.7	4,523,928	2,062,312	144.3	1,307,120	615,116	46.3	160,604
Kansas	3,312,848	1,634,983	259.1	5,998,208	3,017,866	367.6	2,792,849	1,396,439	178.9	2,728,642
Kentucky	1,829,848	910,529	47.4	3,574,038	1,787,019	96.2	1,318,356	659,178	60.0	2,472,207
Louisiana	902,986	445,939	15.9	12,238,713	3,168,308	54.3	1,389,039	686,831	37.8	2,878,612
Maine	1,311,088	649,700	28.6	718,171	359,085	21.1	41,700	32,600	.6	597,411
Maryland	786,000	390,500	18.9	3,354,558	1,671,362	32.5	507,303	253,651	5.9	1,183,412
Massachusetts	1,498,045	746,429	17.5	2,522,214	1,255,454	18.5				3,027,056
Michigan	3,151,089	1,509,738	117.0	10,423,610	5,111,205	274.7	902,863	396,432	33.7	335,045
Minnesota	3,628,763	1,758,810	328.5	6,481,807	3,234,231	361.3	1,381,445	690,105	38.2	2,606,685
Mississippi	1,518,991	580,278	54.6	6,943,874	3,226,931	351.3	1,450,160	702,330	90.8	1,291,597
Missouri	3,165,629	1,573,816	161.6	7,680,516	3,584,161	197.4	3,765,580	1,436,092	105.4	3,020,480
Montana	3,915,103	2,216,637	272.6	2,153,049	1,215,006	122.1	651,061	367,394	36.0	3,063,212
Nebraska	2,984,528	1,486,863	338.8	5,280,008	2,541,429	636.8	1,920,721	960,360	190.3	1,867,119
Nevada	1,404,181	1,208,112	74.6	1,177,081	1,025,149	48.7	209,733	182,842	16.3	573,042
New Hampshire	907,231	442,710	23.2	954,426	470,599	22.3				906,552
New Jersey	1,237,400	618,410	9.3	4,500,040	2,250,020	32.5	1,031,840	515,920	5.5	1,622,731
New Mexico	1,898,611	1,170,006	155.2	1,348,345	812,782	67.9	252,213	152,034	.4	970,165
New York	8,776,249	4,297,359	150.7	13,719,049	6,829,473	185.5	517,127	254,309	9.1	249,838
North Carolina	3,685,799	1,841,737	193.9	5,038,447	2,503,410	217.1	697,390	345,970	36.3	1,078,519
North Dakota	1,655,681	895,171	156.4	2,745,786	1,525,757	223.6	2,626,264	1,333,750	225.6	3,149,602
Ohio	2,518,056	1,258,803	35.9	13,274,952	6,613,147	119.5	6,125,468	3,037,254	48.1	2,568,555
Oklahoma	2,107,265	1,117,698	102.8	2,827,360	1,492,876	79.9	1,201,735	584,342	46.9	3,899,877
Oregon	2,991,449	1,788,011	148.4	1,867,670	1,090,876	36.7	1,191,489	569,740	30.3	564,360
Pennsylvania	3,829,928	1,887,523	54.6	13,121,040	6,509,866	119.4	3,782,878	1,886,212	23.3	893,215
Rhode Island	687,015	342,750	6.4	1,521,497	759,562	14.7				773,451
South Carolina	1,067,566	512,162	74.2	2,192,300	1,054,798	161.3	1,428,090	618,016	66.8	1,884,801
South Dakota	2,974,088	1,663,953	506.5	3,749,903	2,324,163	458.2	978,880	605,380	144.1	2,054,399
Tennessee	1,967,156	978,362	50.3	3,319,190	1,659,595	113.7	553,962	276,981	16.1	3,659,660
Texas	6,334,671	3,113,213	399.7	7,753,942	3,843,752	360.0	4,385,669	2,122,285	186.6	4,333,085
Utah	888,002	644,357	72.0	995,004	745,683	40.8	570,340	297,500	13.4	547,029
Vermont	1,163,682	581,760	36.6	621,508	308,948	15.9	378,009	189,004	8.7	200,755
Virginia	1,659,645	782,545	51.6	3,992,213	1,853,212	74.2	1,026,698	490,849	14.9	979,364
Washington	2,853,173	1,478,309	63.1	2,723,180	1,443,235	43.9	57,006	40,400	.3	672,307
West Virginia	1,465,280	729,175	44.6	3,051,584	1,519,591	85.3	546,060	273,030	7.6	1,494,543
Wisconsin	4,573,933	2,232,152	157.9	2,652,735	1,315,565	110.0	729,051	348,003	13.1	3,083,048
Wyoming	1,664,525	1,044,903	176.4	1,146,942	731,208	139.8				671,761
District of Columbia	345,488	172,744	2.7	363,742	173,837	2.8	539,519	238,200	.7	138,587
Hawaii	95,930	44,205	1.7	614,037	313,171	8.9	243,633	129,161	3.9	1,475,498
Puerto Rico	49,013	24,135	.5	1,675,709	829,135	27.7	196,527	96,495	3.0	621,005
TOTALS	119,068,103	61,061,200	5,595.1	218,040,313	107,321,369	6,763.6	60,407,019	29,514,885	2,199.0	82,273,604

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 190,944	$ 95,263	9.4	$ 1,303,357	$ 649,158	60.7	$ 5,000	$ 5,000		$ 276,238
Arizona	97,773	49,619	10.5	264,890	191,235	13.9				220,478
Arkansas	360,154	159,187	14.6	306,339	152,958	20.6	147,738	73,462	12.2	31,766
California	666,141	362,320	30.3	498,100	270,463	12.3	176,865	92,100	5.0	701,337
Colorado				264,057	148,305	5.8	39,261	22,127	1.2	119,790
Connecticut	287,102	139,108	3.3	188,388	90,213	3.1				165,242
Delaware	69,537	34,768	7.8	104,843	44,274	7.7				265,201
Florida	12,030	6,015		649,252	299,153	14.7	550,715	241,531	5.8	81,912
Georgia	79,007	38,969	10.1	738,149	354,074	62.2	441,783	220,892	43.5	1,000,215
Idaho	111,793	68,954	20.4	193,866	116,204	10.1	87,650	13,350	3.7	105,338
Illinois	1,489,074	737,507	77.5	1,101,200	535,600	29.5	293,600	130,500	16.4	127,321
Indiana	458,671	224,658	31.0	126,882	63,441	5.5	173,600	86,800	10.4	858,468
Iowa	2,017,784	965,451	430.3	808,570	383,540	196.9	329,734	143,693	110.1	53,528
Kansas	268,725	134,166	47.4	918,594	462,205	44.0	216,986	108,493	17.0	1,130,938
Kentucky	747,573	248,935	61.5	398,932	114,095	14.4	499,151	125,751	21.3	254,327
Louisiana	67,133	33,567	8.1	230,796	115,343	17.6				460,035
Maine	287,862	137,140	16.5	51,596	25,798	2.0				5,551
Maryland	94,300	47,150	3.5	87,390	43,695	4.7	86,000	42,500	6.1	383,032
Massachusetts	287,314	142,027	6.0	350,681	174,338	7.5	37,950	18,975	.4	476,543
Michigan	1,046,399	514,992	76.6	733,040	366,520	64.3	470,120	235,060	38.7	351,943
Minnesota	438,119	214,615	52.7	927,189	463,594	150.8	311,889	155,945	37.7	763,867
Mississippi	172,962	86,481	10.6	717,352	353,176	39.0	366,200	160,615	14.3	435,223
Missouri	630,479	314,837	79.1	193,532	96,766	24.3	341,368	135,145	36.3	614,885
Montana	641,506	362,577	80.3	82,618	46,621	3.7	202,812	111,045	44.1	536,900
Nebraska	360,685	181,971	65.6	661,481	322,458	87.3	163,632	81,816	28.8	205,900
Nevada	176,020	147,191	37.1	136,697	118,675	11.4	63,619	55,463	6.8	12,457
New Hampshire	143,639	68,883	3.4	2,192	1,096		83,441	40,100	3.8	127,711
New Jersey	319,500	159,750	10.6	318,057	158,940	11.4				503,134
New Mexico	101,564	59,154	13.1	634,137	343,277	28.8				91,225
New York	1,518,945	719,923	50.3	1,852,301	926,151	57.6				64,956
North Carolina	637,905	317,844	60.9	535,663	269,773	46.8	155,240	72,820	14.6	218,163
North Dakota	42,880	24,583	.3	169,224	90,702	3.6				1,014,689
Ohio	957,917	478,893	33.7	2,370,790	1,184,135	76.2	294,520	147,260	13.4	677,442
Oklahoma	624,887	331,437	43.8	244,665	128,986	14.6	172,130	90,442	12.8	849,896
Oregon	372,237	205,770	56.4	205,058	77,934	13.4	182,677	88,260	18.2	168,370
Pennsylvania	1,198,605	591,779	45.8	1,125,327	561,755	24.1	255,492	127,746	5.0	147,841
Rhode Island	157,358	78,624	3.4	89,358	44,679	.2	84,274	42,080	.9	44,667
South Carolina	211,510	85,994	30.0	552,493	209,873	62.8	504,867	185,200	48.7	72,895
South Dakota				28,926	19,392	9.0				1,265,145
Tennessee	139,047	67,467	10.9	193,926	96,963	7.2	101,734	50,867	2.8	865,132
Texas	1,161,661	568,592	168.8	952,228	460,613	76.0	500,200	242,122	51.5	568,720
Utah	54,999	34,100	9.2	215,530	138,660	22.4	61,685	14,729	2.5	110,203
Vermont	294,751	98,834	11.3	247,776	71,330	9.4				4,391
Virginia	330,770	153,923	20.0	408,412	185,070	15.3	204,652	101,655	9.3	184,715
Washington	476,066	250,888	28.2	192,524	103,039	3.9	235,379	125,900	24.5	114,034
West Virginia	301,750	150,350	16.7	35,469	17,734	1.8	127,225	63,612	4.5	411,469
Wisconsin	232,565	115,254	3.3	698,927	349,360	26.6	202,796	100,130	7.8	555,947
Wyoming	433,925	253,344	42.8	146,623	90,685	9.1				75,261
District of Columbia	61,300	30,650	.7	69,384	34,192	.7				22,703
Hawaii	275,662	137,588	8.6	1,096	1,096					159,971
Puerto Rico				302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	21,108,530	10,431,092	1,862.4	23,630,102	11,714,977	1,448.9	8,227,173	3,780,326	682.2	18,037,523

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PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

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HOUSE DOCUMENT NO. 462

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Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

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MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

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The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

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DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T . . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 21,828	$ 21,828			2	$ 723,776	$ 703,700	7		2	$ 26,439	$ 26,439		1	5	$ 916,112
Arizona	192,342	184,976	3			179,037	178,688	1			19,260	19,260			4	232,120
Arkansas	357,939	357,794	3	1	7	973,893	969,672	10			144,553	144,553	1		9	332,818
California	439,428	439,428	4		6	635,043	456,561	1	1		422,200	420,083	6		13	1,323,225
Colorado						288,868	288,868	1			3,401	3,401			1	915,509
Connecticut	270,224	264,463	1	1		517,556	512,317	6								449,334
Delaware	65,760	65,760			12	124,808	124,808	1		4	13,839	13,839	4		1	460,294
Florida	207,524	203,025	2		2	102,816	102,291	1	1		139,739	139,739			25	1,209,730
Georgia	166,493	166,398	4		12	1,031,420	1,031,420	10	6		317,285	317,285	1	3	6	1,748,913
Idaho	212,890	209,461	5		5	31,172	31,172			7	62,584	62,584			23	411,509
Illinois	637,771	573,151	4	1	32	2,315,032	2,099,250	9	1	66	172,493	158,668			55	1,963,597
Indiana	479,293	479,293	3	1	44	702,019	702,019	4	1	42	45,928	45,928			14	995,474
Iowa	409,261	349,623	4		60	212,449	180,733	2		26	187,279	185,287	2		18	1,069,359
Kansas	679,019	678,912	9		12	301,380	300,902	3	1	4	404,040	404,040	7	1	11	869,617
Kentucky	170,478	169,935	4		11	1,037,019	1,037,019	11			39,110	39,110	1		4	545,571
Louisiana	95,496	95,496	1			345,122	291,627	2			630,384	572,654	11		1	802,109
Maine	159,759	158,841	1	1	6	19,574	19,574		1		125,990	125,990	1			251,016
Maryland	180,997	180,993	1		2	476,609	444,816	2	2	5	15,600	15,600			3	767,315
Massachusetts	15,710	15,710				343,592	333,170	1	1							2,018,040
Michigan	991,140	957,826	7	1	20	1,504,623	1,504,623	3	4	9	77,972	77,972		1	12	639,067
Minnesota	777,244	769,387	8	1	8	1,333,117	1,333,117	9	5	10	124,232	124,232		1	2	1,009,373
Mississippi	68,960	68,960	1			604,334	604,334	9	1		253,300	253,300	2		4	503,812
Missouri	779,990	779,991	3	5	1	2,138,903	1,683,483	8	1		62,344	30,678		1		1,006,711
Montana	427,675	427,675	5		2	82,097	82,096	1			9,155	9,155			3	350,584
Nebraska	224,026	222,355		1	7	821,838	821,838	12	1	3	238,849	238,849	4		10	241,722
Nevada	14,934	14,934			13	125,527	125,527	2		6	75,953	75,953	1		6	56,527
New Hampshire	100,989	100,953	3			149,458	148,638	3		1						325,194
New Jersey	269,185	269,185	2			595,071	595,071	3	1	3	266,607	266,607	1		1	924,243
New Mexico	140,504	140,504	1		6	304,092	274,896	4								482,206
New York	1,030,420	1,011,325	7	3		3,184,529	3,118,563	5	17		244,245	240,245		1	8	2,864,258
North Carolina	472,772	472,709	7	2	13	707,589	707,349	7	4	7	167,200	167,200		2	28	725,619
North Dakota	461,480	461,295	5		19	396,280	396,280	4		4						529,158
Ohio	630,858	609,096	5		14	2,203,465	2,154,275	9	3	2	961,257	933,494	6		1	2,089,292
Oklahoma	260,776	259,749	2	1	26	673,757	668,994	11		3	118,180	118,180			40	1,883,430
Oregon	208,639	117,537	3			254,208	197,981	1			5,790	5,790			2	357,935
Pennsylvania	943,046	934,003	12	1		1,990,424	1,986,163	12	1		1,132,399	1,132,399	8			2,836,327
Rhode Island	3,831	3,750				192,501	192,501		1							94,451
South Carolina	362,765	362,765	4	3	8	150,518	150,115	1		12	255,617	255,017	2		32	820,477
South Dakota	129,470	129,470	2	1	4	583,109	582,249	14			101,060	85,110	2			744,090
Tennessee	204,265	201,038	1	2		173,264	173,264	1	1		33,594	33,594		1		1,720,327
Texas	1,268,167	1,263,359	11		8	1,131,928	1,121,563	10	1	1	218,610	202,630	4		2	1,805,573
Utah	20,001	19,790			5	43,620	42,878	2		10	113,915	113,915			46	229,303
Vermont	97,595	97,595		1	2	151,541	151,541		1	7	8,182	8,182			3	213,187
Virginia	90,201	89,865	1	2	5	687,479	686,149	6		1	135,932	135,932		2	9	593,616
Washington	238,261	238,261	3	1	5	235,287	233,787	2	2	1	419,115	411,513	2	2	6	72,446
West Virginia	5,400	5,400			2	229,982	229,982	2			70,190	68,100	2		1	1,076,437
Wisconsin	814,012	799,461	6	3	7	473,301	444,271	3	1	4	114,346	104,781	1		10	1,274,601
Wyoming						562,886	562,885	6			4,997	4,997			2	166,634
District of Columbia						59,061	59,061	1								176,925
Hawaii						198,846	198,839	2		1	4,684	4,684			1	292,509
Puerto Rico						584,007	579,336	11								414,857
TOTALS	15,798,818	15,443,325	148	33	388	32,887,827	31,620,256	236	60	241	7,987,849	7,796,969	69	16	422	43,802,553

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 11

January 1941

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

APPLICATIONS OF AUTOMATIC TRAFFIC RECORDER DATA IN HIGHWAY PLANNING

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by L. E. PEABODY, Senior Highway Economist, and O. K. NORMANN, Associate Highway Economist

Automatic traffic recorders are being used in 46 States to obtain continuous records of traffic flow at more than 500 locations. Analysis of data from typical stations throughout the United States shows that there was an extremely wide variation in the ratios between traffic for maximum days or hours and the average annual daily traffic, but this variation was considerably less at locations in the southern States as compared with those in northern States. Traffic on the maximum day was normally 233 percent and the maximum hour 25.4 percent of the annual average daily traffic. It is uneconomical to design the average highway for a greater hourly volume than the value exceeded during the 30 peak hours each year, and little will be saved in the construction cost and a great deal lost in expediting the movement of traffic if the design will not handle the volume exceeded during the 50 peak hours. However, the variation in traffic flow between locations is such that detailed data are necessary for a complete engineering analysis of the traffic facilities required at any particular location.

Tests of various schedules of operation that are used in highway planning surveys indicate that eighteen 8-hour counts properly scheduled throughout the year produce results within practical limits of accuracy; that the short count schedule by single hours, or shorter periods, is not as accurate, and its use is limited to relatively compact areas such as a city where time loss and cost of travel may be reduced.

A study of the invariance in seasonal and other types of traffic variation over a period of several years provides a measure of the limitations in the use of factors in estimating annual traffic from observations covering but a small period of time, possibly a few hours. The rather remarkable uniformity in such factors provides considerable confidence in the accuracy of the estimates.

As the traffic records accumulate they will permit an analysis of the traffic trends at a large number of points widely distributed throughout the country and should furnish data useful in setting up regional or national business indices. The relatively brief record now available has already proved of value in making estimates of traffic increases on major segments of the highways and streets of the Nation.

ONLY a few years ago information with regard to the volume of motor-vehicle traffic by hours throughout the year was available only at a few bridges, where it was obtained incidental to the collection of tolls. Usually these data were summarized and reported only as an annual total or, at best, subtotals were obtainable by months. The need for complete traffic flow information was recognized, but until the State-wide highway planning surveys were begun no concerted effort was made to obtain such data on anything approaching a Nation-wide scale. Today, at more than 500 points throughout the country, and in nearly every State, traffic-flow information is being obtained by means of permanently installed traffic counters. At many hundreds of additional points this information is collected by means of portable traffic counters. The cost of this mechanical counting is but a small fraction of the cost of manual counting.

Since detailed traffic data are so recently available and the record correspondingly short, it is quite certain that not all of the practical uses of these data have developed. Indeed the record is so short that an adequate study of some applications of the data is not possible. Nevertheless, it is clear that the principal uses of these data are as follows: (1) In measuring the time during which a section of highway is congested, and the fraction of the year's traffic that moves under conditions of congestion; (2) in compiling a traffic record, obtained under widely varying climatic, geographic, and economic conditions, essential in planning extensive traffic surveys such as those forming a part of the highway planning surveys, and in which some traffic information is obtained for every mile of publicly used highway; (3) in acquiring knowledge of the variations in traffic volume required in expanding short traffic counts covering but a small fraction of the year to obtain reasonably accurate estimates of the traffic total for the year; and (4) the traffic record is of vital importance in the study of traffic trends and their relationships to economic factors and to probable future traffic. It is mainly with regard to the latter use that the record is inadequate; and this deficiency is being reduced as the records accumulate with each passing month and year.

The automatic traffic counters used in the State-wide highway planning surveys are of two general types; one designed to be installed permanently at key locations and referred to as a fixed-type recorder; and a portable counter used in obtaining short counts at a large number of widely separated locations.

AUTOMATIC RECORDERS YIELD HOURLY RECORDS OF TRAFFIC FLOW

The fixed-type machine[1] is much larger, more expensive, and more dependable than the portable traffic counter. The fixed-type machines are designed to count passing vehicles without counting pedestrians. Two parallel beams of light approximately 30 inches center to center, directed across the roadway upon photoelectric cells, must be interrupted simultaneously to operate the counting mechanism. Pedestrians, who interrupt only one beam at a time, are not counted by the machine. Every hour, on the hour, these machines stamp on a record tape the day, hour, and cumulative counter reading, thus producing an hourly record of the number of vehicles passing the location. The cost of one of these machines is approximately $400 and the average cost of installation is approximately $125 per machine. A survey of the operating costs in 1938 for all States using this equipment gave the average cost of operating one fixed-type automatic recorder at a rural location for a month as follows:

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[1] The May 1938 issue of PUBLIC ROADS carries a detailed description of these machines.

Overhead		$4.87
Supervision		3.51
Maintenance:		
Labor	$7.64	
Subsistence	1.78	
Travel	8.86	
Power	4.36	
Supplies, etc.	2.88	
Total maintenance		25.52
Preparation of records		10.09
Grand total		43.99

The portable-type traffic counters used in the planning surveys consist of two general types, the recording counter and the cumulative or nonrecording counter.[2] The recording-type machine produces records by printing or photographing the cumulative counter reading on a record tape every hour on the hour. The cumulative counter enables a record to be obtained only of the total traffic passing the machine between readings by an observer. In a few instances, cumulative counters have been equipped with a clock that starts and stops the machine at predetermined times, thus eliminating the necessity for placing the machine and picking it up at a definite time.

The operating mechanisms of the portable counters are of two types electrically operated and mechanically operated. The majority of the mechanically operated machines are an adaptation of a watch or clock, so arranged that the escapement is operated when the wheels of a vehicle pass over the detector. These counters have generally been referred to as watch-type counters. So far this type of construction has been confined to cumulative counters. However, work is in progress to develop a recording counter that will be entirely spring operated.

Most of the portable machines now in operation make use of a pneumatic detector consisting of a rubber tube placed across the roadway and a diaphragm of some flexible material at one end of the tube. The air impulse produced when each pair of wheels of a vehicle passes over the tube causes the diaphragm to move, which, in turn, either actuates the contacting elements controlling the counting circuit, or operates directly the escapement of the counting mechanism. Other detectors used with portable machines are a photoelectric device using one light beam, and a positive-contact device consisting of two strips of spring steel, enclosed in a waterproof casing, which make contact when pressed together by the wheels of a vehicle passing over them.

The cost of portable counters ranges from $10 for the watch-type cumulative counter to $225 for the hourly recording type machine. A number of States have constructed cumulative counters of the electrically operated type at a cost of approximately $25 per machine, all of which use the pneumatic detector. One State has constructed recording counters using the pneumatic detector at a cost of approximately $80 per machine. Another State has constructed a portable counter using a photoelectric detector with a photographic recording device at a cost of approximately $125 per machine.

The portable traffic recorders have not been in use a sufficient length of time for the cost of their operation to have been accurately determined. One difficulty in determining the cost of records is that it depends almost entirely on the distance between stations and the schedule upon which they are operated. One State has reported a total cost of $1.62 per count for 24-hour counts obtained with the simple cumulative counter. This cost includes salary, mileage, parts, power, and incidentals. Another State reports a total cost of approximately 87 cents per 24-hour count. These figures are for eastern States where stations are close together. The estimated monthly cost of operation (parts, batteries, and incidentals) is $4 for one of the recording-type portable machines. The operating cost of the cumulative counters is less than that, so it is very evident that the charges for salary and mileage are the major part of the cost of counting traffic with portable traffic counters.

Experimental development and field tests of the automatic traffic counters were carried on throughout 1935, and 84 of the fixed-type machines were placed in operation by the States during 1936. In 1937, 115 additional fixed-type counters were installed; in 1938, 120; in 1939, 168; and up to July 1940, 45 new fixed-type machines were placed in operation. A total of 532 such machines were in operation during July 1940. A complete statement of the record, by States, is given in table 1, and the locations of these machines are indicated in figures 1, 2, and 3.

Locations for the machines were chosen by the States with the assistance of the Public Roads Administration. Detailed knowledge of economic areas within the States and of the character of traffic using individual routes were factors in the selection of locations. Consequently, farm-to-market roads, roads used largely by tourist traffic, and roads upon which intercity commercial traffic is a considerable fraction of total traffic, are included among the locations.

To obtain information regarding the fluctuation of traffic flow on primary highways, automatic traffic counter records for 90 stations located on the main U. S. numbered highways have been analyzed. In selecting record stations for analysis, an attempt was made to include scattered locations so that the figures for annual traffic volumes would cover a wide range and be geographically distributed throughout all sections of the United States. The traffic records for each of the selected stations show the number of vehicles for almost each hour during at least 1 full year.

FLUCTUATIONS IN TRAFFIC FLOW GREATER IN NORTH THAN IN SOUTH

Table 2 shows the location, the period used for the analysis, and the annual average 24-hour traffic volume for each of the stations. Stations located in 43 States and having annual average 24-hour traffic volumes ranging from 311 to 13,624 vehicles were used.

Figure 4 shows the maximum 24-hour traffic volume at each location during the year, plotted against the annual average 24-hour traffic volume. For any annual volume, there is a large variation in the peak day during the year. For example, the roads with an annual average of about 4,000 vehicles per day have from 6,000 to 18,000 vehicles on the peak day, or a variation of 300 percent. The average shown by the solid line indicates that there is a slight drop from a straight-line relationship as the volume increases. For sections that have annual averages between 2,000 and 4,000 vehicles there is a marked sag in the curve. On an average, the maximum 24-hour traffic volume was 2.45, 2.20, and 2.34 times the annual average 24-hour volume for locations with annual averages below 2,000, between 2,000 and 4,000 and over 4,000 vehicles, respectively.

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[2] A simple counter of this type is described in the January 1939 issue of PUBLIC ROADS.

TABLE 1.—*Number of automatic traffic counters*[1] *which started operation in various years*

State	1936	1937	1938	1939	1940
Alabama		9		1	
Arizona	7				
Arkansas			11	5	
California		10			
Colorado	1		2	3	
Connecticut				20	
Florida	6		4		2
Georgia				12	
Idaho	4				
Illinois			1	5	
Indiana	4			14	
Iowa	2		10	12	
Kansas	1		3		
Kentucky		4	2	5	
Louisiana		2	2	4	
Maine			6		
Maryland		10	1	2	
Massachusetts			8		1
Michigan	1	8	1		
Minnesota	9	2		16	6
Mississippi				10	
Missouri		5	7	5	1
Montana			6	8	
Nebraska	5		2		
Nevada	1	7	2	2	1
New Hampshire		3			
New Mexico	9		1		
New York			12	9	
North Carolina			4		
North Dakota	3		2	4	
Ohio	2		5		10
Oklahoma	9				11
Oregon	2	3			
Pennsylvania	1	20	1	1	7
Rhode Island		4			
South Carolina		1	6	6	
South Dakota		5			3
Tennessee		4			
Texas	4	10	2	14	1
Utah	2		4		2
Vermont		1		3	
Virginia			4		
Washington	3	7			
West Virginia	4			7	
Wisconsin	4		8		
Wyoming			3		
Total	84	115	120	168	45
Cumulative total	84	199	319	487	532

[1] Machines operating in July 1940.

TABLE 2.—*Location of automatic traffic recorders used to obtain data for study of fluctuation in traffic density*

State	Location		Period used		Annual average 24-hour traffic volume
	State's recorder station No.	United States route No.	From—	To—	
Alabama	2	72	1- 1-39	12-31-39	531
	4	78	12-25-37	12-24-38	1,073
Arizona	1	60, 70, 80	7- 7-39	7- 6-40	7,174
	4	60 and 89	1-28-39	1-27-40	1,743
Arkansas	11	63	1- 1-39	12-31-39	311
California	1	99	7-10-37	7- 9-38	5,815
	2	99	2-20-37	2-19-38	2,281
Colorado	3	85-87	2-27-37	2-26-38	4,334
	11	85	6-26-38	6-25-39	5,472
Connecticut	6 and 7	Merritt Parkway	3-31-39	3-30-40	13,624
	17	5	3-31-39	3-30-40	8,313
Florida	1	90	11-27-38	11-26-38	749
	3	41	1- 1-38	12-31-38	1,668
	4	90	5-15-37	5-14-38	3,365
Georgia	1	41 and 411	1- 1-39	12-31-39	3,238
	12	84	1- 1-39	12-31-39	632
Idaho	1	10	1- 1-38	12-31-38	2,438
	2	30	4- 3-37	4- 2-38	3,085
	3	30	1- 1-38	12-31-38	2,290
Illinois	1	45	9-27-36	9-26-37	4,057
	2	66	1-24-37	1-23-38	3,937
	7	50	12-18-37	12-17-38	3,210
Indiana	2A	20	8-28-37	8-27-38	3,490
	42A	52	7- 3-37	7- 2-38	3,071
	59A	40	1-15-38	1-14-39	3,125
	72A	31	1-15-38	1-14-39	2,293
Iowa	601	65-69	12-19-36	12-18-37	3,290
	601	65-69	1- 1-38	12-31-38	3,539
Kansas	3	50S	2-18-39	2-17-40	2,059
	5	24 and 40	8-14-38	8-13-39	2,183
Louisiana	1	79-80	12-25-37	12-24-38	3,304
	4	90	4-24-37	4-23-38	4,226
Maine	2	1	2- 5-38	2- 4-39	1,287
Maryland	2	40	4- 3-37	4- 2-38	3,030
	12	40	1-22-38	1-21-39	7,250
Massachusetts	8	1	4-30-38	4-29-39	7,363
	10	6	7-21-39	7-20-40	6,176
Michigan	676	27	10- 2-37	10- 1-38	3,151
	678	23	1- 1-39	12-31-39	1,200
Minnesota	157	212-169	3-20-37	3-19-38	4,875
	159	10-52 and 169	9-11-37	9-10-38	3,730
	175	52	7- 3-37	7- 2-38	872
Missouri	5	54	7-17-37	7-16-38	1,708
	9	66	1-23-39	1-22-40	5,220
Wyoming	205	20	5-19-39	5-18-40	1,309
	204	30	1- 1-39	12-31-39	1,257
Montana	A4	10-12	10-29-38	10-28-39	982
	A7	91	6-30-39	6-29-40	495
Nebraska	2	30	1- 8-38	1- 7-39	1,619
	5	6	1- 8-38	1- 7-39	2,128
Nevada	101	40	11- 6-37	11- 5-38	1,469
	107	40	6- 5-37	6- 4-38	755
New Hampshire	1	3	9-18-37	9-17-38	1,360
New Mexico	1	85-285	6-12-37	6-11-38	1,216
	6	66	1-15-38	1-14-39	1,574
	7	70-80	8- 7-37	8- 6-38	1,461
	9	54-70	1- 8-38	1- 7-39	751
New York	5-1	5 [1]	12-31-38	12-30-39	4,458
North Carolina	3	29	1- 1-39	12-31-39	4,296
	4	19 and 23	2-25-39	2-24-40	2,540
North Dakota	102	1 [1]	2- 1-39	1-31-40	356
	103	2	10-18-37	10-17-38	352
Ohio	25	42	4-12-39	4-11-40	3,645
	27	25-68	2-18-39	2-17-40	3,928
Oklahoma	1	66-69	5-15-37	5-14-38	2,111
	5	77	2-27-37	2-26-38	2,259
Oregon	Rowena (3).	30	11-27-37	11-26-38	1,261
Pennsylvania	1	20	11-20-37	11-19-38	4,395
	4	6	7-24-37	7-23-38	1,231
Rhode Island	2	1-A [1]	6- 4-38	6- 3-39	1,931
South Carolina	2	15-52	12- 4-37	12- 3-38	1,583
	105	29	2-20-37	2-19-38	3,936
South Dakota	101	14-16	5-15-37	5-14-38	982
	106	18	12-31-38	12-30-39	479
Tennessee	1	31W	4-21-39	4-20-40	3,425
Texas	1	80	7- 7-39	7- 6-40	9,053
	4	77-81	1- 1-38	12-31-38	4,049
	5	80	12-19-36	12-18-37	2,427
	8	81-83	3-20-37	3-19-38	875
Utah	301	40	11-13-37	11-12-38	1,766
	302	50-91	7-10-37	7- 9-38	3,443
Vermont	A-12-2	2	11-28-36	11-27-37	1,615
Virginia	1	1	6-26-37	6-25-38	6,668
	4A	58	1-31-39	1-30-40	2,429
Washington	1	99	12-28-37	12-27-38	3,590
	3	99, 410, 101	9-11-37	9-10-38	3,385
	4	99	12-11-37	12-10-38	3,479
	10	10	4-10-37	4- 9-38	3,233
Wisconsin	2 and 3	41	1- 8-38	1- 7-39	5,614
	10	10 and 12	1- 9-37	1- 8-38	1,632

[1] State route.

An investigation of the surface width at each location showed that all stations with annual averages below 3,400 vehicles had 2 traffic lanes. As the annual average increased above 3,400, the relative number of sections wider than 2 lanes increased until at 4,500 vehicles practically all sections had more than 2 lanes. It, therefore, seems that the sag in the curve was due to a tendency for some drivers to avoid heavily traveled 2-lane highways on days of peak traffic.

A classification of the stations by their geographic location showed that at stations in the North, where there usually is considerable snow and ice each winter

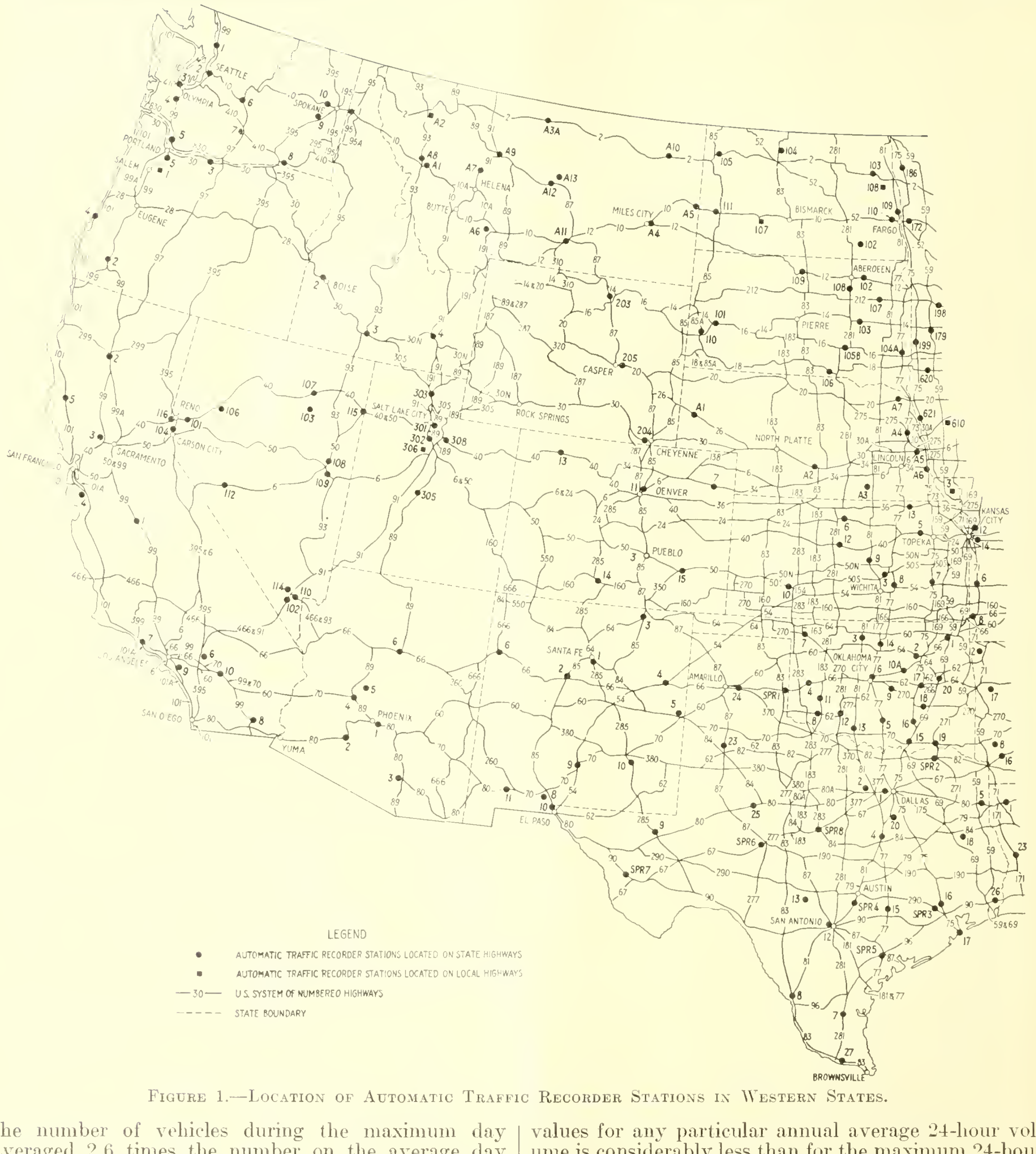

Figure 1.—Location of Automatic Traffic Recorder Stations in Western States.

the number of vehicles during the maximum day averaged 2.6 times the number on the average day while in the South there were only 1.8 times as many vehicles on the maximum day as on the average day. The curves for both the northern and southern locations (fig. 4) show the same general tendency for the slope of the curves to decrease when the annual volume reaches about 2,000 vehicles per day and then to increase and return nearly to the former slope at between 4,000 and 4,500 vehicles per day.

Figure 5 shows the tenth highest 24-hour traffic volume for each station plotted against the annual average 24-hour volume. The variation in the tenth highest values for any particular annual average 24-hour volume is considerably less than for the maximum 24-hour volume. On an average, the traffic volume on the tenth highest day is 1.75 as great as the annual average 24-hour volume. Corresponding figures for the locations in the northern and southern States are 1.88 and 1.44, respectively. In other respects, the curves are very similar to those for the maximum days. The tenth highest day was selected as an index because it is felt that it represents the conditions that should be expected on an average Sunday in summertime.

Figure 6 shows the same average curves as those presented in figures 4 and 5, together with curves for

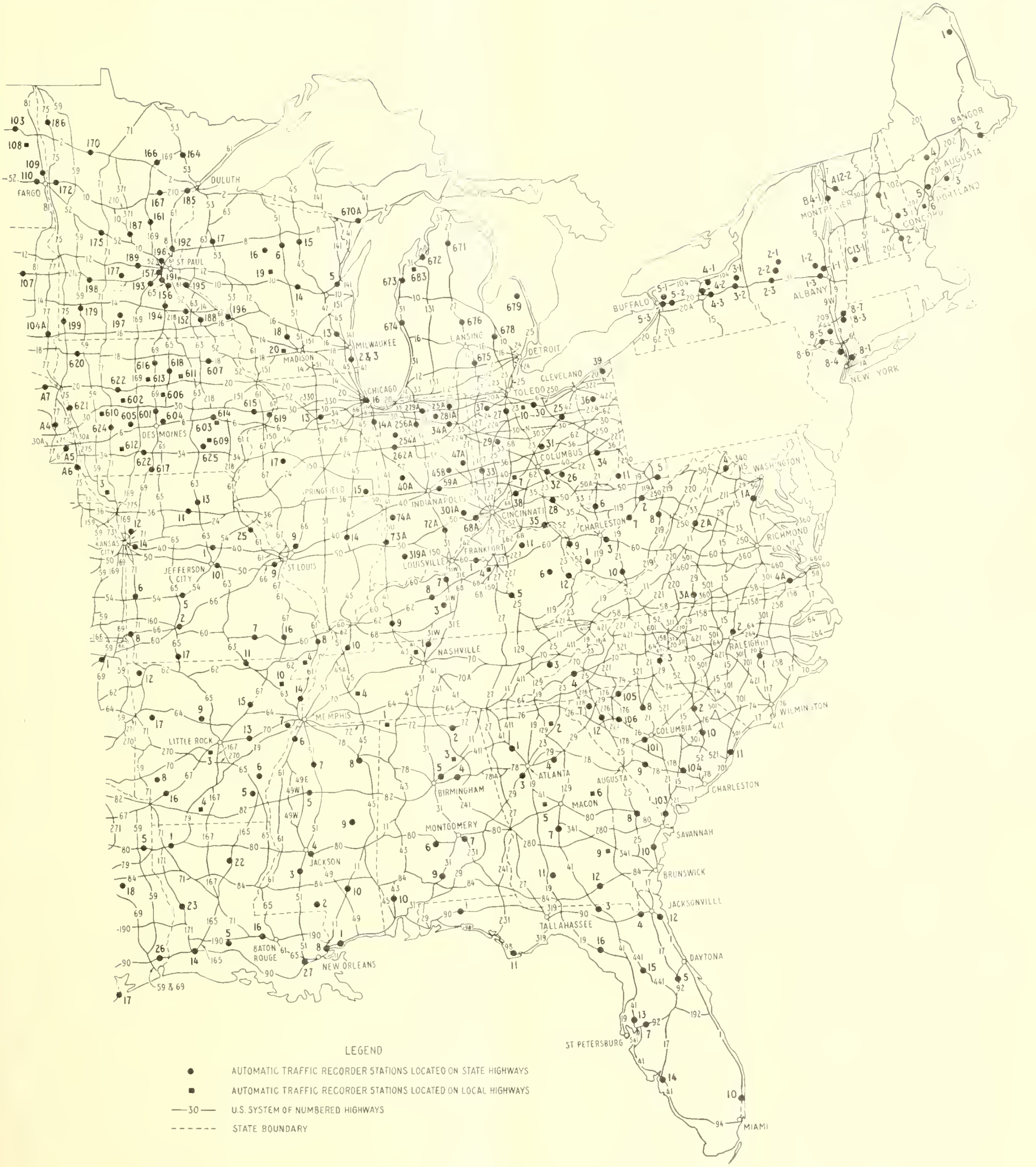

Figure 2.—Location of Automatic Traffic Recorder Stations in Eastern States.

the average 24-hour volumes during the maximum week and maximum month. The slope of each of the curves as obtained from the original data decreased slightly when the annual average reached about 2,000 vehicles and then increased until at an annual average of about 4,500 the former slope was nearly reached. Since the reason for this was probably due to congested conditions on a number of the roads in this group, the relations shown by the curves on figure 6 are more useful when considering design features to accommodate the various traffic volumes. However, figure 7 illustrates that even these curves are of little value in determining maximums from the annual average since there is a large variation between different stations. For example, although the volume on the maximum day for the average location is 2.32 times as high as the volume on

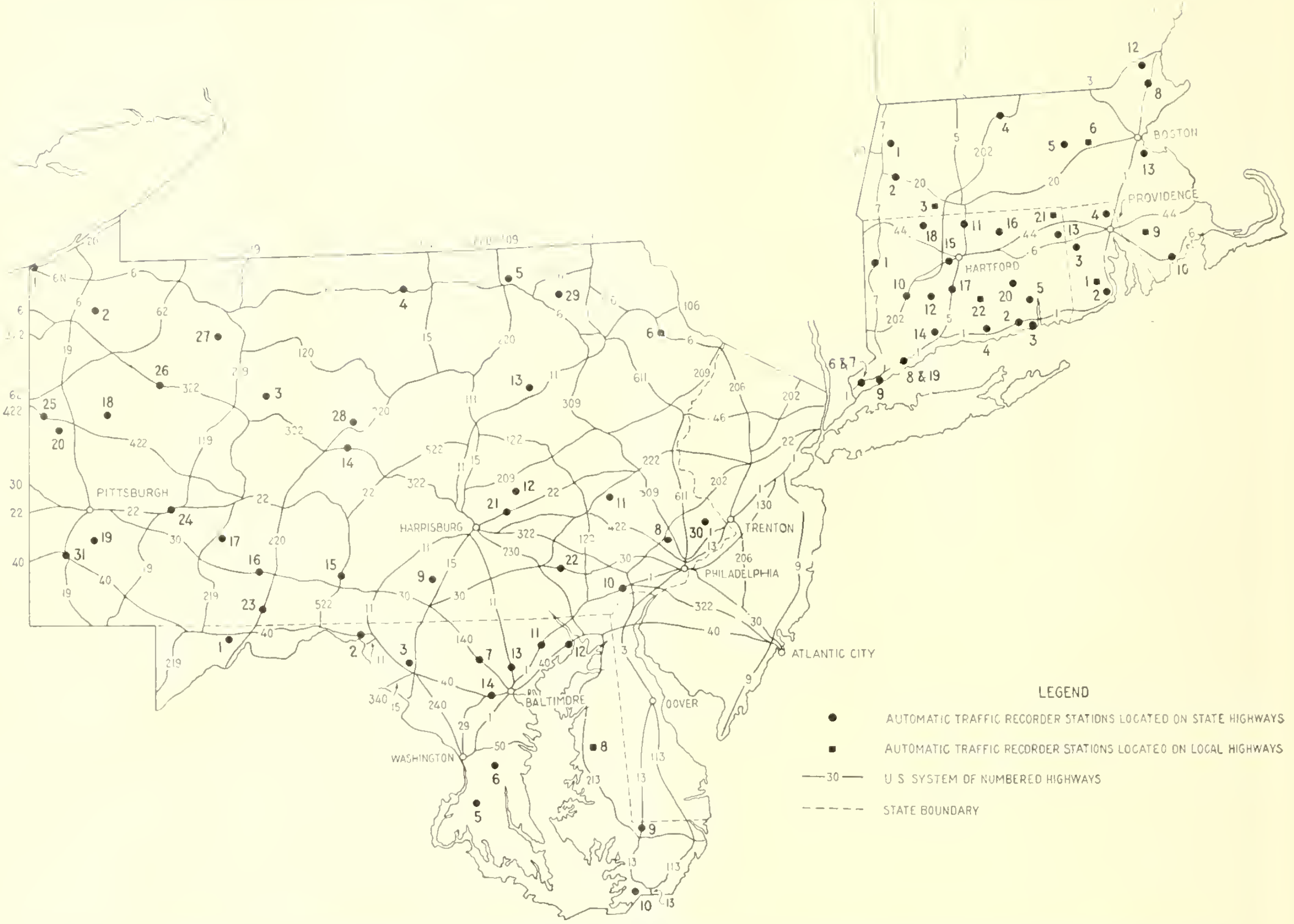
BOSTON
PROVIDENCE
HARTFORD
PITTSBURGH
HARRISBURG
TRENTON
PHILADELPHIA
ATLANTIC CITY
BALTIMORE
DOVER
WASHINGTON
LEGEND
● AUTOMATIC TRAFFIC RECORDER STATIONS LOCATED ON STATE HIGHWAYS
■ AUTOMATIC TRAFFIC RECORDER STATIONS LOCATED ON LOCAL HIGHWAYS
—30— U S SYSTEM OF NUMBERED HIGHWAYS
- - - - - STATE BOUNDARY

FIGURE 3.—LOCATION OF AUTOMATIC TRAFFIC RECORDER STATIONS IN 7 EASTERN STATES NOT SHOWN IN FIGURE 2.

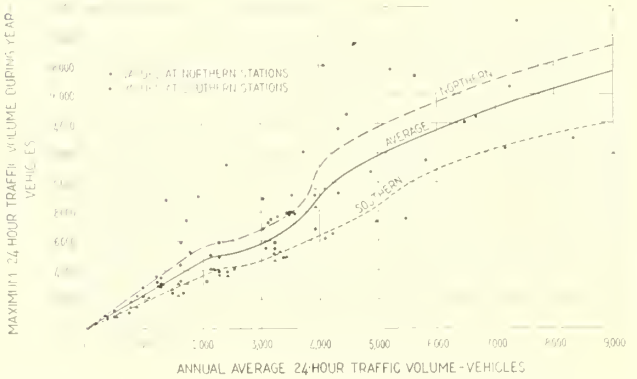

FIGURE 4.—MAXIMUM 24-HOUR TRAFFIC VOLUME DURING 1 YEAR FOR VARIOUS ANNUAL AVERAGE 24-HOUR TRAFFIC VOLUMES.

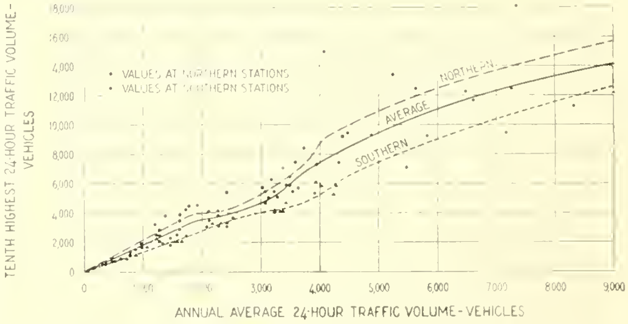

FIGURE 5.—TENTH HIGHEST 24-HOUR TRAFFIC VOLUME DURING 1 YEAR FOR VARIOUS ANNUAL AVERAGE 24-HOUR TRAFFIC VOLUMES.

the average day, the group ranging from 1.4 to 1.8 includes a larger percentage of the locations than any other group covering a similar range. In all cases, the maximum values for the southern stations do not cover as great a range as the northern stations and the values for the southern stations are closer to the annual averages.

Figure 8 shows, for different annual average 24-hour traffic volumes, the average number of days during a year that the traffic volume exceeded various values. Thus, highways with an average of 6,000 vehicles per day on an annual basis carried over 12,000 vehicles on 3 days, over 11,000 vehicles on 11 days, over 8,000 vehicles on 50 days, etc. The curves shown on the figure indicate that for the average location, the 24-hour traffic volume that is exceeded any certain number of days is nearly proportional to the annual average 24-hour traffic volume.

LARGE PROPORTION OF TRAFFIC MOVES DURING PEAK HOURS

Thus, the average highway carrying 4,000 vehicles a day has approximately the same number of days per year with a traffic volume in excess of 5,000 vehicles as a highway carrying 8,000 vehicles per day has days when traffic exceeds 10,000 vehicles. The curves show 56 days in the one case and 47 days in the other.

Since for all roads there is a large variation in the traffic volumes for different hours of the day, and since

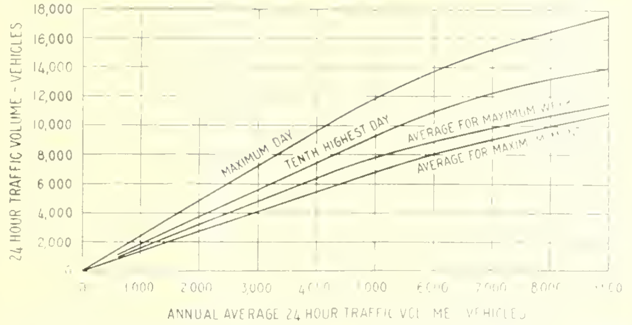

Figure 6.—Relation Between Various 24-Hour Traffic Volumes During Year and Average 24-Hour Traffic Volume. (Determined From Data for 89 Highway Locations.)

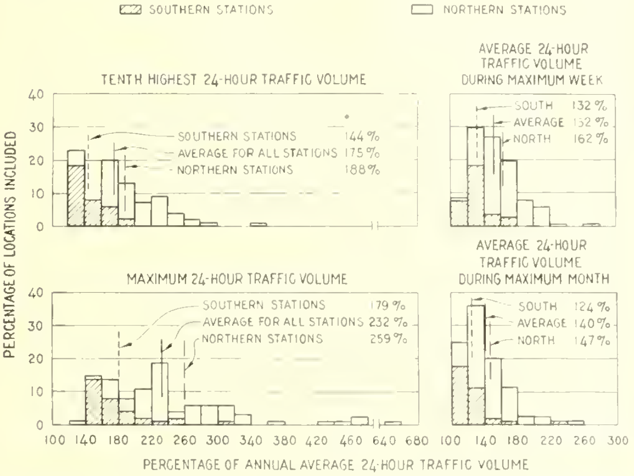

Figure 7.—Variation in Relation Betwfen 24-Hour Traffic Volumes During Peak Traffic Density Periods and Annual Average 24-Hour Traffic Volumes at Different Locations.

the hourly rather than the daily volume is the more practical unit to use as a basis for measuring the capacity of a highway and for design purposes, a number of figures showing the relations between the annual average 24-hour volumes and the individual hourly volumes are presented.

Figure 9 shows the relations between the maximum hour during a year and the average 24-hour volume at each location. The range in maximum hours for stations having similar yearly traffic volumes is great. There are cases in which the maximum for one highway is nearly six times as great as the maximum for another highway carrying the same total number of vehicles during a year. Even the fiftieth highest hour as shown by figure 10 is sometimes three times as high for one station as for another station with the same annual traffic.

The slopes of curves for the relations between the maximum and fiftieth highest hours and the annual 24-hour averages also have a tendency to decrease when the annual average reaches about 2,000 vehicles and then to increase until they return almost to their former slopes near 4,000 vehicles per hour. The curves for the stations located in northern States are considerably higher than those for the stations in southern States.

Figure 11 shows the relations between the maximum hour, the tenth, thirtieth, and fiftieth highest hours, and the average daily volume during the year. The curves shown in this figure have been smoothed to eliminate the sags at annual volumes between 2,000 and 4,000 vehicles which were probably caused by conges-

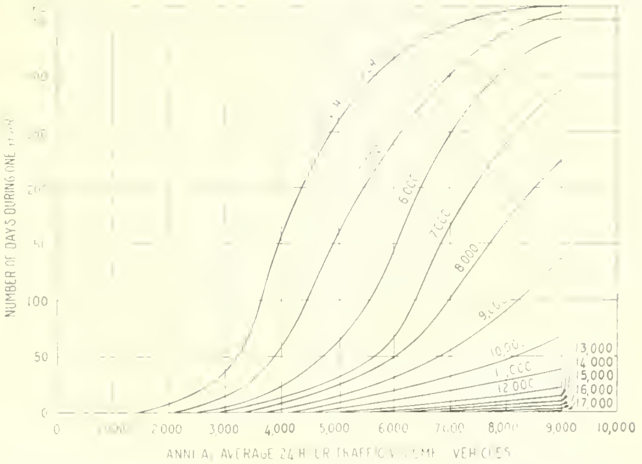

Figure 8.—Number of Days During 1 Year That Various 24-Hour Traffic Volumes Were Exceeded. (Determined From Data for 89 Highway Locations.)

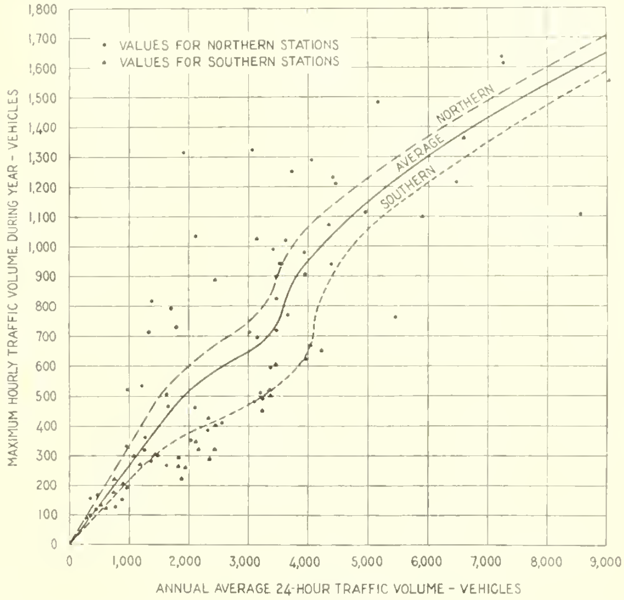

Figure 9.—Relation Between Maximum Hourly Traffic Volume During Year and Annual Average 24-Hour Traffic Volume.

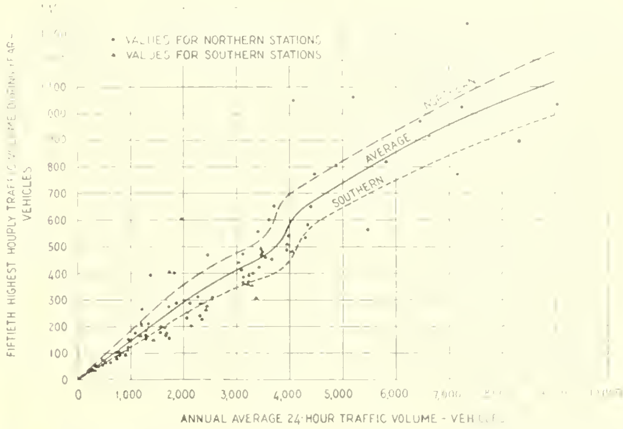

Figure 10.—Relation Between Fiftieth Highest Hourly Traffic Volume and Annual Average 24-Hour Traffic Volume.

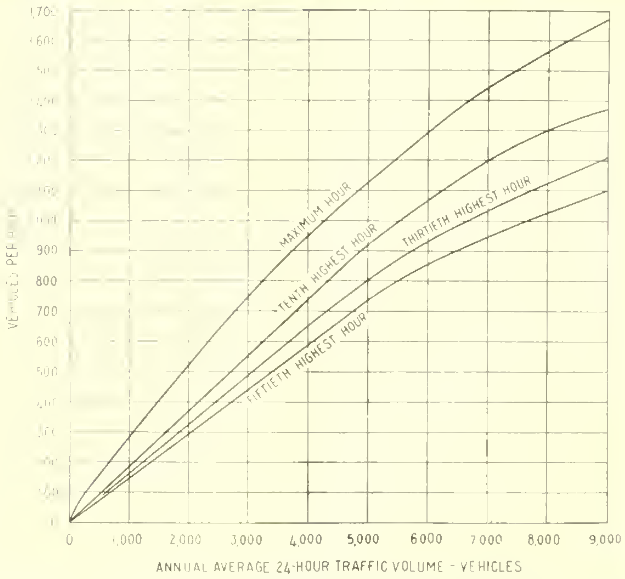

Figure 11.—Relation Between Various Hourly Traffic Volumes During Year and Annual Average 24-Hour Traffic Volumes. (Determined From Data for 89 Highway Locations.)

tion on some of the 2-lane roads in this group during peak hours.

The variations in the percentages that the peak hourly volumes are of the annual average 24-hour volumes for different locations are shown by figure 12. It may be seen from this figure that the variation between locations decreases as the number of peak hours that are included increases. Thus, although the maximum hours average 25.4 percent of the average daily volume, at only 23.5 percent of the locations is the maximum between 20 and 25 percent of the annual average. For 69 percent of the locations the fiftieth highest hour falls within the same 5-percent range group as the average for all of the fiftieth highest hours. As with the daily volumes, the peak hourly volumes for the northern locations cover a wider range and are a larger percentage of the annual average 24-hour density than corresponding peaks for southern locations.

Data were available for the percentages that out-of-State and commercial vehicles were of the total traffic for 70 of the 90 locations studied. There did not seem to be any relationship between the percentage of out-of-State vehicles and the traffic volume fluctuation but, on an average, there was a slight decrease in the fluctuation with an increase in the percentage of trucks (table 3). Since in the automatic counter records there is no separation of trucks from passenger cars, it was not possible to determine the cause of this decrease. It is reasonable to assume that the peak truck densities occur at different times, either seasonal, daily, or hourly, than the peak passenger-car densities. Furthermore, routes of heavy truck traffic are usually those between centers of population between which also flows a substantial volume of passenger cars used on weekdays for business purposes. Both of these factors tend to increase the weekday volume in comparison to the Sunday flow.

Table 4 shows the relation between the number of vehicles during peak traffic density periods and the annual average 24-hour traffic volume. On an average, there is a very rapid decrease in the average hourly volume during the peak period as the number of hours included in the peak period is increased. When the 50 hours of peak traffic density, which cover only 0.57 percent of the total time, are included, the average hourly volume is only 16.6 percent of the annual 24-hour average whereas the maximum hour is 25.4 percent of the annual 24-hour average. The percentage of total vehicles included in the peak hours is always relatively large as compared to the percentage of time involved.

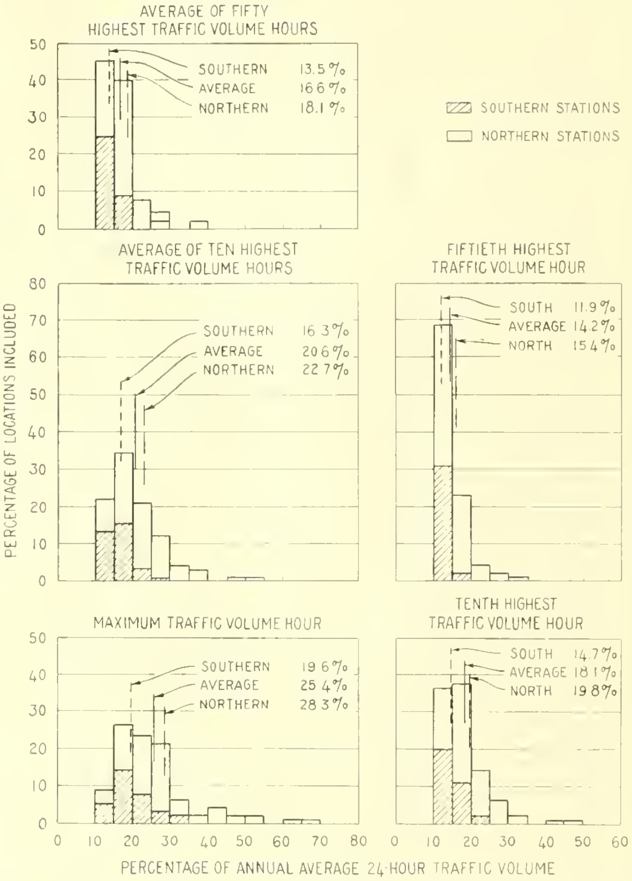

Figure 12.—Variation in Relation Between Hourly Traffic Volumes During Peak Hourly Traffic Density Periods and Annual Average 24-Hour Traffic Volumes at Different Locations.

Table 3.—*Effect that the percentage of trucks has on the relation between the traffic volume during peak density periods and the annual average 24-hour volume*

Percentage of trucks included in total traffic		Number of locations	Percentage of annual average 24-hour traffic volume		
Group limits	Average		Maximum hour during year	Tenth highest hour during year	Fiftieth highest hour during year
Below 15	10.9	9	27.7	21.2	15.9
15–20	17.4	19	26.2	18.4	14.6
20–25	22.6	22	26.4	18.2	14.2
Above 25	27.6	20	23.2	17.3	13.6

DATA ON TRAFFIC VOLUMES DURING PEAK HOURS NEEDED FOR DESIGN OF HIGHWAYS

Figure 13 shows the average number of hours each year that the traffic density exceeded various hourly traffic volumes for highways with different annual average 24-hour volumes. Thus, highways carrying an annual average of 5,000 vehicles per day had 610 hours when the traffic volume exceeded 400 vehicles per hour, 350 hours when the traffic volume exceeded 500 vehicles per hour, 200 hours when the traffic volume exceeded 600 vehicles per hour, etc.

It is a generally accepted fact that it is not economically advisable to construct a highway to accommodate the peak traffic densities that will use it during its probable life, unless to do so involves no additional construction cost over designs to accommodate fewer vehicles. However, the time, percentage of time, number of vehicles, or percentage of vehicles that may be included in the peak traffic densities not cared for by the design are still unknown quantities. Although the design will depend to a large extent upon the funds available for construction, figure 13 throws some light on the hourly traffic volumes for which highways with different annual traffic densities and having average traffic fluctuations should be designed. From the figure, it may be seen that for any annual average 24-hour traffic volume, there is a rapid increase in the number of hours included between each 100-vehicle change in the hourly volume when the number of hours included is greater than the 50 peak hours, but there is only a small change in the number of hours included as the volume goes below the value shown for the thirtieth highest hour.

Table 4.—*Relation between number of vehicles during peak traffic density periods and the annual average 24-hour traffic volume (average for 60 northern and 30 southern stations)*

Time period	Percent that average hourly traffic volume during peak density periods is of annual average 24-hour traffic volume			Percentage of total time included (annual basis)	Percentage of total annual traffic included		
	Northern stations	Southern stations	All stations		Northern stations	Southern stations	All stations
Maximum month (30 days)	6.1	5.2	5.8	8.21	12.03	10.26	11.44
Maximum week	6.8	5.5	6.3	1.92	3.13	2.53	2.90
10 highest days	8.9	6.4	8.1	2.74	5.85	4.21	5.33
Maximum day	10.8	7.4	9.7	.27	.71	.49	.64
Maximum hour	28.3	19.6	25.4	.01	.08	.05	.07
10 highest hours	22.7	16.3	20.6	.11	.62	.45	.56
20 highest hours	20.9	15.0	19.5	.23	1.15	.82	1.07
30 highest hours	19.6	14.3	18.2	.34	1.61	1.18	1.50
40 highest hours	18.8	13.9	17.4	.46	2.06	1.52	1.91
50 highest hours	18.1	13.5	16.6	.57	2.48	1.85	2.30

For example, at the average location with an annual average 24-hour traffic volume of 4,000 vehicles, the various hourly traffic volumes are exceeded for the number of hours shown in the following tabulation:

Hourly traffic volume:	*Number of hours during 1 year*
950	1
800	8
700	20
650	30
600	50
500	115
400	280

A design based on the maximum hourly volume would be required to handle nearly 1½ times as many vehicles per hour as a design based on the 30 peak traffic volume hours, but the additional number of vehicles accommodated would only be 1.5 percent of the annual traffic (table 4). On the other hand, designing for a traffic volume only 30 percent less than the volume exceeded during 50 hours would result in a 560 percent increase in the number of hours of traffic not accommodated by the design. The percentage of the total number of vehicles using the highway that would not be accommodated by the design would be increased from 2.3 to 9.9. It, therefore, seems that for the average highway, it is impractical to design for a greater hourly volume than the value which will be exceeded only during the 30 peak hours each year and that little will probably be saved in the construction cost and a great deal lost in expediting the movement of traffic if a design is used that will not handle the traffic volume exceeded during the 50 peak hours. The exact value to use depends upon the traffic volumes that the different designs will accommodate. Thus, if the traffic volume is such that to accommodate the hourly volume exceeded for 30 hours during a year requires a greater number of traffic lanes than to accommodate the hourly volume exceeded for 50 hours, the lower number of lanes should probably be used.

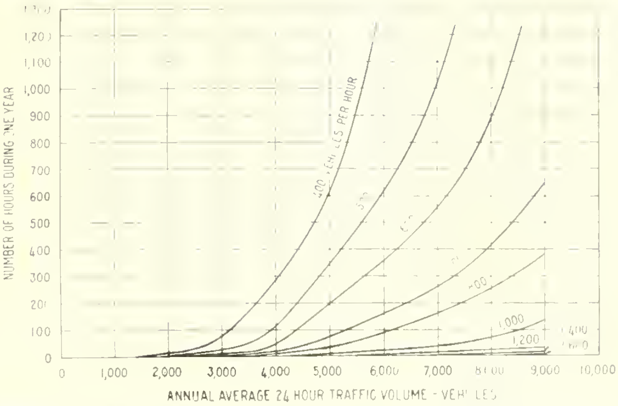

Figure 13.—Number of Hours That Various Hourly Traffic Volumes Are Exceeded on Highways Having Different Annual Traffic Densities. (Determined From Data for 89 Highway Locations.)

Since this analysis is based on the average fluctuation in traffic density for many highways, the results are not applicable to each location. For an extreme example, a comparison has been made of the data for the station included in this anlaysis that had the greatest fluctuation in the hourly traffic volumes during the year and the station that was found to have the most uniform flow of traffic. The percentage of the total time during which each of these road sections carried traffic volumes in excess of different numbers of vehicles per hour and the percentage of all vehicles that passed over each road section when the hourly traffic volume was in excess of the specified traffic densities are shown by table 5. The section with the largest variation in traffic flow had an annual average 24-hour traffic volume of 4,057 vehicles, was located in the North, and is referred to as section A. The one with the most uniform traffic flow had an annual average 24-hour traffic volume of 4,226 vehicles, was located in the South, and is referred to as section B.

Although practically the same number of vehicles used these two road sections in 1 year, the traffic on section B was rarely in excess of 500 vehicles per hour, while on section A it sometimes reached 1,200 vehicles per hour and was in excess of 500 vehicles per hour for 5.5 percent of the time. Since the percentage of the total vehicles during high density periods is greater than the percentage of the total time occupied by the same density periods, 25.1 percent of the vehicles traveled over section A during the 5.5 percent of the time that the hourly density exceeded 500 vehicles.

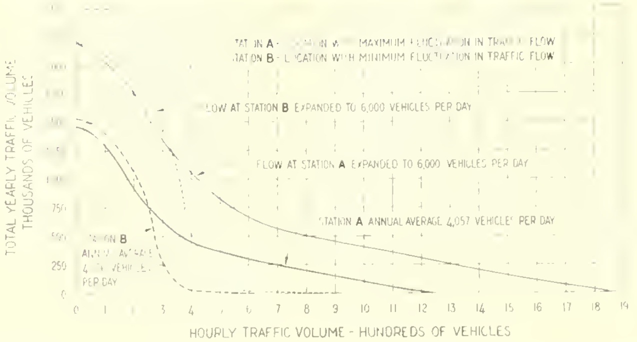

Figure 14.—Cumulative Frequency Curves Showing the Number of Vehicles When Traffic is in Excess of Various Hourly Traffic Volumes at Stations Having Maximum and Minimum Fluctuation in Traffic Flow.

Table 5.—*Percentage of time and percentage of vehicles included during periods that road sections carried traffic in excess of different hourly volumes*

Hourly volume, vehicles	Cumulative percentage of total time		Cumulative percentage of total vehicles	
	Section A	Section B	Section A	Section B
1,200	0.2		1.3	
1,100	.5		3.3	
1,000	1.1		6.8	
900	1.6		9.9	
800	2.3		13.3	
700	3.0		16.4	
600	4.2	(1)	20.8	0.1
500	5.5	0.1	25.1	.2
400	7.6	.9	30.3	2.3
300	14.0	9.0	43.2	18.0
200	26.7	46.5	61.2	70.2
100	57.6	71.6	87.5	91.0
0	100.0	100.0	100.0	100.0

[1] Less than 0.1 percent.

Figure 14 shows the data obtained from the automatic traffic recorders located at these 2 stations in a most useful form. The curve for traffic at station B shows that a highway designed to accommodate 400 vehicles per hour would be the most economical design at this location for the present traffic, since designing for a greater volume would result in but a slight increase in the number of vehicles accommodated, and designing for a traffic volume even slightly less than 400 vehicles per hour would result in a relatively large increase in the number of vehicles that would be required to use the highway during periods when the volume was in excess of the design value.

RECORDS OF PAST YEARS USEFUL IN ESTIMATING FUTURE PEAK TRAFFIC VOLUMES

Highway design for the traffic flow at station A presents a more difficult problem. Based only on the annual traffic density, the same design could be used at both locations; but if the design at station A were based on 400 vehicles per hour, nearly half a million, or one-third of the vehicles, would use the road during periods when the traffic density exceeded the design value. A design to accommodate the same percentage of vehicles at station A as are accommodated by a design of 400 vehicles per hour at station B would have to accommodate 1,200 vehicles per hour. The actual design value for the location represented by station A would depend entirely upon the funds available and the hourly capacity of highways of different designs. However, if the present width of surface and alinement were identical at these two locations, the highway with the traffic flow represented by station A should be given prior consideration in any construction or improvement program designed to reduce traffic congestion such as the elimination of short sight distances, increasing the surface width, increasing the number of traffic lanes, or providing grade separations.

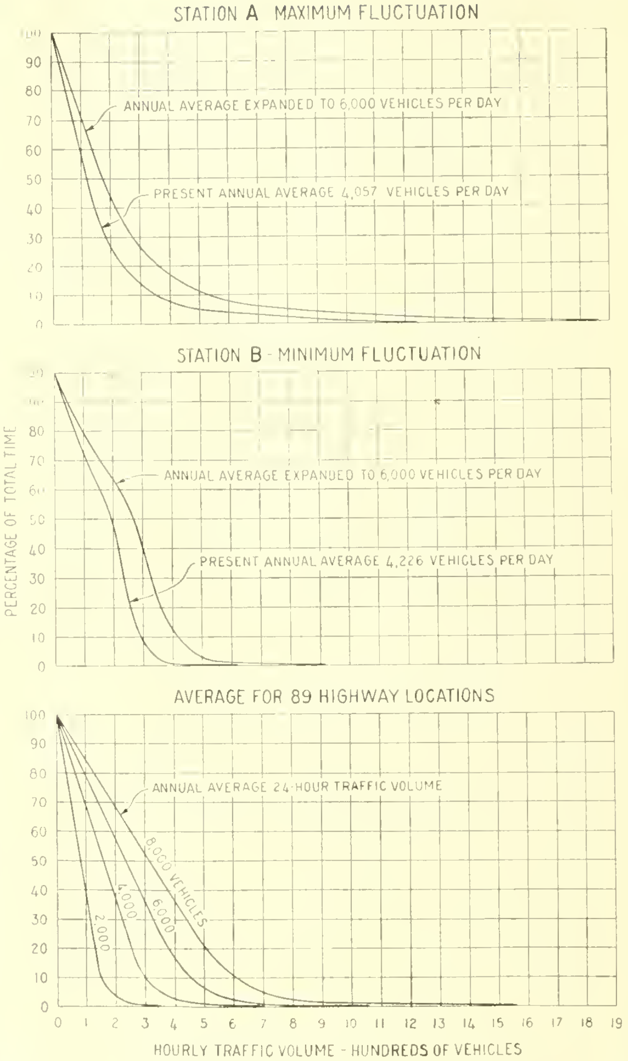

Figure 15.—Cumulative Frequency Curves Showing the Percentage of Time That the Traffic Was in Excess of Various Hourly Volumes on Highways Having the Maximum, Minimum, and Average Fluctuation in the Flow of Traffic.

Since highway construction programs must be based on future as well as present traffic densities to avoid obsolescence in a relatively short time, it is essential to estimate future fluctuations in the traffic volumes as well as the future increase in the annual traffic. A study of the future variation in traffic flow can usually be based on the present fluctuation. When a cumulative frequency curve such as the one shown in figure 14 has been determined, it will generally be safe to assume that the shape of the curve will not change materially with either an increase or decrease of average daily traffic unless it is definitely known that some local development will tend to alter the shape of the curve.

If it is assumed that an increase in the annual traffic affects all portions of present traffic volumes proportionally and that the annual average daily traffic will increase to 6,000 vehicles at some future date, the

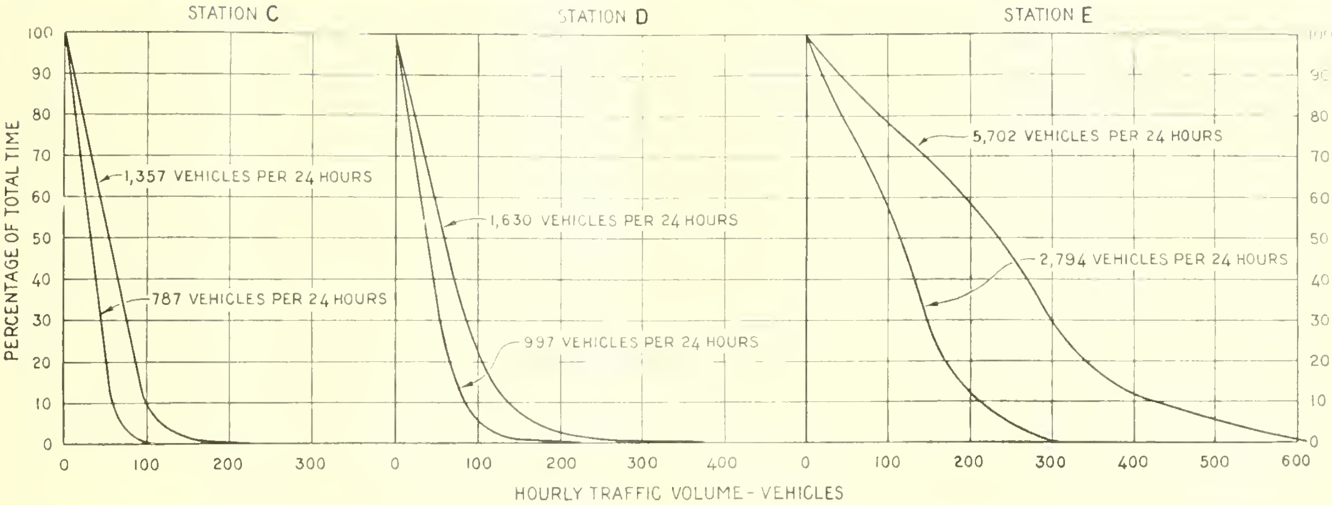

Figure 16.—Percentage of Time That Traffic Was in Excess of Various Hourly Densities at Stations Where There Was an Appreciable Difference in the Average 24-Hour Volumes for the Same Period in Successive Years.

cumulative frequency curves as obtained from the present records made by the automatic recorders can be expanded by increasing both values for points along the present traffic curve in the same ratio as the estimated future annual traffic is to the present traffic. By expanding the curves for the present traffic on sections A and B in this manner to annual average daily volumes of 6,000 vehicles, the expanded cumulative frequency curves as shown by the light lines on figure 14 were obtained. In a similar manner, the data for the present traffic can be expanded to any annual average daily volumes. It is interesting to note that at the present time, with a volume of 4,057 vehicles per day, a larger number of vehicles travel over the highway represented by station A during periods when the traffic volume exceeds any value over 420 vehicles per hour, than will travel over the highway represented by station B when the annual average daily volume reaches 6,000 vehicles.

Since the curves shown in figure 14 represent locations with the maximum and minimum fluctuation in traffic flow found by analyzing records at 90 stations located on U. S. routes in all parts of the country, it is reasonable to expect that similar curves for practically all sections on U. S. numbered highways will fall somewhere between the curves representing these two locations for corresponding annual traffic volumes. However, the range between the two curves for identical traffic volumes is so great that they emphasize the importance of having at least a full year's record from an automatic traffic recorder before an intelligent analysis can be made of the traffic needs on any particular section of highway where improvements to increase the traffic capacity of the highway are contemplated.

Cumulative frequency curves of the type shown in figure 15 are useful when it is desired to compare the percentage of time that traffic on different road sections is in excess of various hourly volumes. The data obtained from the automatic traffic counters at the stations included in this analysis where the maximum and minimum fluctuation in traffic flow were recorded, have been used in plotting the curves for stations A and B, respectively. When expanding the data shown by the original curves to other traffic volumes, the values along the abscissa are increased by the same ratio as the annual traffic, while the values along the ordinate are held constant. The values for stations A and B have been expanded to show the percentage of time that the traffic will be in excess of various hourly volumes when the annual average volume increases to 6,000 vehicles per day (fig. 15). In a similar manner, the data for 89 locations were expanded to annual 24-hour traffic volumes of 6,000 vehicles and the values averaged to obtain the average cumulative frequency curve shown in figure 15. This curve and other curves formed by expanding the individual values to other traffic densities show the relation between time and hourly traffic density for highways with the average fluctuation in traffic flow.

The method outlined above for estimating the percentage of time, number of vehicles, or number of hours included in the various hourly traffic density groups when there is a change in the annual traffic, assumes that the change will affect all portions of the cumulative frequency curves proportionately. This will always be true when all portions of the traffic pattern are affected proportionately but may also be true even though there is a material change in the traffic pattern.

Since automatic, hourly recording counters have only been in operation during recent years, there were only three stations where the recorders had been operated continuously for at least 2 years and where there had been sufficient increases in the annual traffic densities during the period of operation to check the accuracy of this assumption. At these three locations, referred to as stations C, D, and E, the total traffic volumes during the same period in successive years had increased from averages of 787, 997, and 2,794 vehicles per day to 1,357, 1,630, and 5,702 vehicles per day, respectively. The cumulative curves for the percentage of time that traffic at the 3 stations was in excess of various hourly volumes during each of the 2 different traffic density periods are shown by figure 16. In each case, if the values shown for the lower volume curve are expanded in the same ratio as the two average 24-hour volumes are to each other, as previously outlined, the curves for the higher average volumes will be exactly duplicated.

While such a close agreement will probably not be found for all locations, especially where local developments tend to influence the traffic pattern and where the increase takes place over a period of 10 or 20 years, the data available at the present time substantiate the

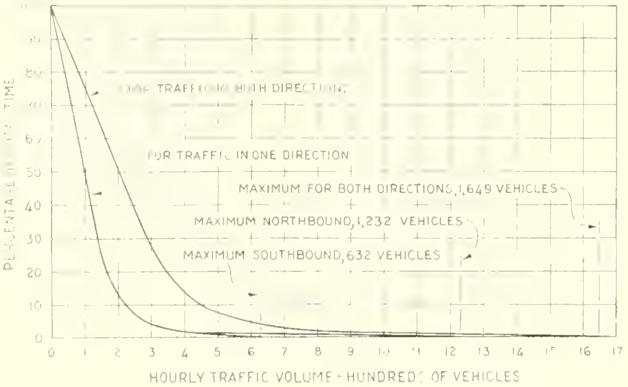

Figure 17.—Percentage of Time That Traffic Density on U. S. Route 41 Was in Excess of Various Hourly Volumes. (Annual Average 24-Hour Traffic Volume Was 5,614 Vehicles.)

one assumption necessary to expand the automatic recorder data to care for increased annual traffic densities.

PERCENTAGE OF TRAFFIC MOVING IN EACH DIRECTION DURING PEAK HOURS IMPORTANT

For design and traffic control purposes it is often desirable to know the percentage of the total vehicles traveling in each direction during hours of high traffic density. This can be obtained for divided highways by using an automatic traffic recorder for each of the two directions. On undivided roadways, the automatic recorders using either light beams or the direct contact or pneumatic tube as the means of detection can be equipped with special units so that only vehicles traveling in one direction will be recorded. Approximate values can also be obtained when the contact type of detector is used by placing the detector so that only vehicles traveling on one-half of the roadway will be recorded. By proper selection of locations, the error due to vehicles traveling to the left of the center of the roadway, as when passing, can be reduced to a minimum.

Cumulative frequency curves for two locations on divided highways, where automatic traffic counters obtained the number of vehicles in each direction for each hour during periods exceeding 1 year, are shown by figures 17 and 18.

The percentage of time that the traffic at automatic recorder stations 2 and 3 on U. S. Route 41, 18 miles south of Milwaukee, Wisconsin, was in excess of various hourly volumes is shown by figure 17. At station 2 south-bound traffic was recorded, while at station 3 north-bound traffic was recorded. By adding the number of vehicles in the two directions for corresponding hours, the total traffic on the route during each hour of the year was obtained. Although the number of vehicles traveling in each of the two directions was rarely the same for any particular hour, the number of hours that each direction carried the various traffic volumes below 300 vehicles per hour was approximately equal to the number of hours during a year that the total traffic volume in both directions was twice the corresponding densities. Both directions carried traffic volumes in excess of 300 vehicles per hour for 4 percent of the time, and the total volume was in excess of 600 vehicles per hour for 4 percent of the time. The maximum volume of south-bound traffic was 632 vehicles per hour and the maximum volume of north-bound traffic was 1,232 vehicles per hour, but the total volume never exceeded 1,649 vehicles per hour. During the 1 hour that the total volume reached 1,649 vehicles, 74.7 percent of the traffic was in one direction. During the 10 peak hours of total traffic volume, the traffic in one direction averaged 70 percent of the total traffic.

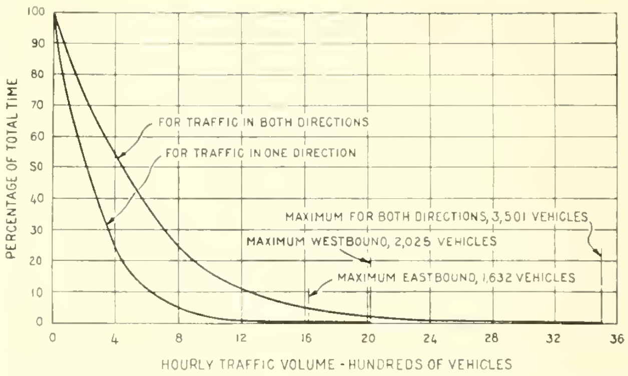

Figure 18.—Percentage of Time That Traffic Density on the Merritt Parkway Was in Excess of Various Hourly Volumes. (Annual Average 24-Hour Traffic Volume Was 13,624 Vehicles.)

On the Merritt Parkway, at traffic recorder stations 6 and 7 near Greenwich, Conn., the traffic in one direction exceeded all traffic volumes below 1,100 vehicles per hour for the same number of hours that the total volume exceeded twice the corresponding densities (fig. 18). East-bound, west-bound, and total traffic never exceeded 1,632, 2,025, and 3,501 vehicles per hour, respectively. During the 10 peak hours, the traffic in one direction averaged 57 percent of the total traffic.

The results obtained at these two locations indicate that if a cumulative frequency curve of the type shown in figures 17 and 18 is available for either the traffic in one direction or for the total traffic, the curves for both the traffic in one direction and the total traffic can be obtained, except for a very small portion of the total time when the peak volumes occur. It is also evident that unless practically all the vehicles are to be accommodated, designs for each direction of traffic based on half of the total volume are sufficient, but if all vehicles are to be accommodated, the design for each direction must in some cases be based on volumes as high as 70 percent of the peak total volumes.

COMPLETE TRAFFIC RECORDS USEFUL IN SELECTING SCHEDULES FOR TRAFFIC SURVEYS

The second of the general problems for which automatic traffic recorder data furnish a means of attack is that of planning observation schedules for traffic surveys. A satisfactory schedule must require sufficient observation in the field to enable an accurate estimate of the year's total traffic and of the various types of vehicle units of which it is composed. Results of the schedule operation should enable the analyst to make estimates of the ranges in traffic volume—in particular an estimate of traffic during periods of maximum volume.

The schedule should be so devised as to balance cost of operation against accuracy of results; i. e., the time for which it is necessary to pay men to count traffic should be as small as possible so that costs will be low, while the time for which traffic must be observed must be as large as is necessary to assure accurate results.

It has been recognized in earlier analyses [3] that traffic volume is affected principally by the hour, day of the week, and the month in which the count is taken. Less predictable effects upon traffic volume result from variation in weather conditions, detoured traffic from a normal route due to road construction or other reasons, holidays, football games, fairs, or other events attracting unusual numbers of people.

There are, of course, a number of means by which allowance may be made in the schedule of operation to provide measures of the hourly, daily, and seasonal fluctuations in traffic. Because of the numerous possibilities, it is feasible to test but a few of these possible schedules. Since total traffic has been measured by the automatic traffic recorders, the average daily traffic may be computed with precision; and since data are available for every hour and every day of the year at a large number of locations, any combination of hours, days, and seasons may be selected. From the selected periods or assumed schedule, an estimated average daily traffic may be computed. Comparison of the estimated values under various assumed schedules with exact values obtained from the year's complete record will establish the relative accuracy of the various schedules selected for test.

One of the schedules selected for test is the "key station schedule" first used [4] in the Western States Traffic Survey and in subsequent surveys in which the Public Roads Administration cooperated, and by the various States in the Highway Planning Surveys.

> Each operation covered a 10-hour period on a staggered schedule from 6 a. m. to 4 p. m. and from 10 a. m. to 8 p. m. with splits in the count at 10 a. m. and 4 p. m. This permitted a continuation series of the 10 a. m. to 4 p. m. section through all operations, which were scheduled to provide two counts for each of the 7 days of the week. Sufficient night counts from 10 p. m. to 6 a. m. were obtained to adjust all data to a 24-hour day.[5]

When the 8-hour counting period became generally used, this schedule was modified to cover the 6 a. m. to 2 p. m. and 2 p. m. to 10 p. m. periods alternately at intervals of 26 days, thus covering each day of the week at 6-month intervals (schedule I). Sufficient night counts, usually four in number, were seasonally spaced to cover the 10 p. m. to 6 a. m. period. The effects of the schedule were: To balance the seasonal variation in traffic; to cover the full 24 hours at each point of observation; to cover each of the days of the week at every point; and to set up the operation in such a manner as to keep a relatively small force of men continuously employed, with days of no work equivalent to those received by men in other forms of employment.

The second schedule (schedule II) to be tested is that recommended at the location of the recording type automatic traffic counters in the continuing traffic surveys conducted as an integral feature of the Highway Planning Surveys. Machines are operated for a 24-hour period on Saturday and Sunday and either on Friday or Monday to give a continuous record of the three typical days of the statistical week. These stations may be considered as control stations of the continuing blanket counts.

The third schedule (schedule III) to be tested with the data available from the automatic traffic recorders is one in which it is assumed that each period of observation is but 1 hour in duration. There are a total of 40 such observation periods at each station, scattered throughout the year as indicated in the following sample schedule:

a. m.		*p. m.*	
12– 1	Apr. 7	12– 1	Mar. 14 Sept. 22
1– 2	May 25	1– 2	Mar. 26 (Sunday) Oct. 4
2– 3	July 12	2– 3	Apr. 7 Oct. 16
3– 4	Aug. 29	3– 4	Apr. 19 Oct. 28 (Saturday)
4– 5	Oct. 16	4– 5	May 1 Nov. 9
5– 6	Dec. 3	5– 6	May 13 (Saturday) Nov. 21
6– 7	Jan. 1 (Sunday) July 12	6– 7	May 25 Dec. 3 (Sunday)
7– 8	Jan. 13 July 24	7– 8	June 6 Dec. 15
8– 9	Jan. 25 Aug. 5 (Saturday)	8– 9	June 18 (Sunday) Dec. 27
9–10	Feb. 6 Aug. 17	9–10	June 30 Jan. 7 (Saturday)
10–11	Feb. 18 (Saturday) Aug. 29	10–11	Jan. 1
11–12	Mar 2 Sept. 10 (Sunday)	11–12	Feb. 18

It will be noted that under this schedule of operation the period from 6 a. m. to 7 a. m. is covered in January and in July, at nearly 6-month intervals. The 7 a. m. to 8 a. m. hour is also covered in January and July, again approximately at 6-month intervals, and so for all of the hours from 6 a. m. to 10 p. m. The remaining hours, those normally of much lesser traffic importance, are covered but once, at approximately 6-week intervals throughout the year.

ACCURACY OF THREE OBSERVATION SCHEDULES DETERMINED

The estimates of average daily traffic under each assumed schedule are computed as follows: At the key stations (schedule I), traffic observed during the 6 a. m. to 2 p. m., 2 p. m. to 10 p. m., and 10 p. m. to 6 a. m. periods is averaged and the three averages are totaled for the estimated average daily traffic. At the control blanket-count stations (schedule II), the observed weekday traffic is multiplied by 5, traffic counts for a Saturday and a Sunday are added, and the total is divided by 7 for the counts taken during each season. The 4 seasonal averages, thus computed, are totaled and divided by 4 to give the estimated average daily traffic for the year. At the stations where traffic is assumed to have been observed only during hourly periods (schedule III) the averages of the 2 observations for each hour from 6 a. m. to 10 p. m. are obtained. To these averages (16 in number) are added the observed traffic for each hour from 10 p. m. to 6 a. m. The result is the estimated average daily traffic under this schedule.

Tables 6, 7, and 8 present the average daily traffic computed from schedules I, II, and III, using the analysis methods outlined above. In table 6 the stations were those located on State routes that carried a relatively large volume of traffic. In table 7, stations were also those located on State routes, but with a light traffic volume; while in table 8 all stations were on local routes, usually those carrying a smaller traffic volume than the stations used in table 7. Thirty-three stations were included in each of the above classes.

In addition to the computed averages, the true average daily traffic and the ratios of the various computed averages to the true averages, are tabulated. Weighted averages of these ratios are shown in the last line of each table.

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[3] Highway Traffic Analysis Methods and Results, by L. E. Peabody. PUBLIC ROADS, vol. 10, No. 1, March 1929.

[4] The Western States Traffic Survey, by L. E. Peabody. PUBLIC ROADS, vol. 13, No. 1, March 1932.

[5] Digest of Report on Arkansas Traffic Survey, by L. E. Peabody. PUBLIC ROADS, vol. 17, No. 6, August 1936.

TABLE 6.—*Automatic traffic recorder averages for year 1939, State routes carrying heavy traffic*

State	Station	Average daily traffic				Percentage of actual average		
		Schedule			Actual average for year			
		I	II	III		I	II	III
Massachusetts	1	2,926	3,413	3,066	2,959	98.9	115.3	103.6
Pennsylvania	22	6,635	7,462	7,062	7,069	93.9	105.6	99.9
Connecticut	2	3,811	4,127	3,755	[1] 3,915	97.3	105.4	95.9
	17	7,993	8,974	8,444	[2] 8,112	98.5	110.6	104.1
Florida	10	3,500	4,356	3,576	3,462	101.1	125.8	103.3
	13	1,748	1,934	1,924	1,805	96.8	107.1	106.6
Michigan	676	3,241	3,926	3,430	3,460	93.7	113.5	99.1
Louisiana	14	2,999	2,977	2,974	3,046	98.5	97.7	97.6
Missouri	9	5,131	5,372	5,278	5,266	97.4	102.0	100.2
Texas	1	8,774	9,130	9,323	9,102	96.4	100.3	102.4
Colorado	11	5,480	5,507	6,010	[3] 5,578	98.2	98.7	107.7
Washington	10	3,270	3,521	3,418	3,427	95.4	102.7	99.7
Oregon	2	1,012	990	989	985	102.7	100.5	100.4
California	1	6,091	6,185	6,452	6,316	96.4	97.9	102.2
	10	4,105	4,464	4,383	4,159	98.7	107.3	105.4
Alabama	5	5,300	5,390	5,755	5,381	98.5	100.2	107.0
	7	1,488	1,592	1,547	1,612	92.3	98.8	96.0
Arizona	1	7,115	7,528	7,592	7,210	98.7	104.4	105.3
	3	1,873	1,967	2,003	[4] 1,889	99.2	104.1	106.0
Arkansas	13	2,191	2,118	2,186	[5] 2,169	101.0	97.6	100.8
	14	2,480	2,540	2,382	[5] 2,542	97.6	99.9	93.7
California	6	2,892	2,652	2,442	[6] 2,637	109.7	100.6	92.6
	2	2,526	2,464	2,465	[6] 2,521	100.2	97.7	97.8
	9	4,015	4,073	3,805	[7] 4,141	97.0	98.4	91.9
Connecticut	12	4,883	5,218	4,809	[4] 5,085	96.0	102.6	94.6
	15	9,015	9,363	9,696	[1] 9,367	96.2	99.9	103.5
Georgia	1	3,249	3,166	3,155	3,238	100.3	97.8	97.4
	3	4,347	4,430	4,260	4,363	99.6	101.5	97.6
Idaho	2	2,677	2,742	2,820	2,724	98.3	100.7	103.5
	3	2,436	2,438	2,430	2,468	98.7	98.8	98.5
Illinois	9	4,314	4,586	4,273	4,465	96.6	102.7	95.7
Indiana	59A	3,179	3,664	3,295	3,407	93.3	107.5	96.7
Iowa	601	3,219	3,774	3,437	[8] 3,444	93.5	109.6	99.8
Weighted average						97.5	103.4	100.8
Weighted percentage of error						3.01	4.28	3.59

[1] Feb. 18, 1939–Feb. 17, 1940.
[2] Feb. 25, 1939–Feb. 24, 1940.
[3] Dec. 17, 1938–Dec. 16, 1939.
[4] Mar. 10, 1939–Mar. 9, 1940.
[5] Jan. 29, 1939–Jan. 28, 1940.
[6] Year 1938.
[7] Feb. 5, 1937–Feb. 4, 1938.
[8] Apr. 16, 1938–Apr. 15, 1939.

TABLE 7.—*Automatic traffic recorder averages for year 1939, State routes carrying light traffic*

State	Station	Average daily traffic				Percentage of actual average		
		Schedule			Actual average for year			
		I	II	III		I	II	III
Arizona	5	201	206	193	206	97.6	100.0	93.7
Arkansas	7	194	209	207	198	98.0	105.6	104.5
Georgia	11	280	295	299	292	95.9	101.0	102.4
Iowa	607	435	455	448	434	100.2	104.8	103.2
Louisiana	13	151	149	153	[1] 150	100.7	99.3	102.0
Minnesota	171	263	268	271	[2] 275	95.6	97.5	98.5
	174	283	293	332	[3] 298	95.0	98.3	111.4
Missouri	7	595	652	611	608	97.9	107.2	100.5
Montana	A-7	462	474	421	474	97.5	100.0	88.8
Nebraska	A-3	208	207	220	213	97.7	97.2	103.3
Nevada	114	263	226	224	228	115.4	99.1	98.2
New Hampshire	3	538	565	437	513	104.9	110.1	85.2
Oklahoma	8	1,091	1,087	1,110	1,111	98.2	97.8	99.9
Pennsylvania	7	302	364	344	[4] 358	84.4	101.7	96.1
Rhode Island	3	325	337	307	326	99.7	103.4	94.2
South Carolina	104	676	687	694	665	101.7	103.3	104.4
Texas	8	863	821	877	848	101.8	96.8	103.4
	9	538	532	504	526	102.3	101.1	95.8
Utah	305	724	783	765	766	94.5	102.2	99.9
Washington	9	230	226	241	222	103.6	101.8	108.6
West Virginia	8	540	556	502	[5] 551	98.0	100.9	91.1
Alabama	6	614	671	701	667	92.1	100.6	105.0
California	4	772	808	736	829	93.1	97.5	88.8
Connecticut	4	716	752	630	[6] 757	94.6	99.3	83.2
Florida	11	393	375	350	381	103.1	98.4	91.9
Kansas	7	898	883	952	909	98.8	97.1	104.7
Kentucky	4	301	310	289	[7] 295	102.0	105.0	98.0
Maine	4	400	414	367	407	98.3	101.7	90.2
Maryland	3	386	421	401	376	102.7	112.0	106.6
Michigan	672	972	1,007	872	969	100.3	103.9	90.0
Pennsylvania	5	498	531	577	543	91.7	97.8	106.3
South Dakota	106	452	468	469	479	94.4	97.7	97.9
Wisconsin	16	892	934	993	998	89.4	93.6	99.5
Weighted average						97.5	100.5	97.8
Weighted percentage of error						4.05	3.12	5.65

[1] Oct. 29, 1938–Oct. 28, 1939.
[2] Aug. 6, 1938–Aug. 5, 1939.
[3] Aug. 20, 1938–Aug. 19, 1939.
[4] Oct. 1, 1938–Sept. 30, 1939.
[5] Mar. 18, 1939–Mar. 17, 1940.
[6] Feb. 18, 1939–Feb. 17, 1940.
[7] Year 1938.

A comparison on the basis of these weighted averages indicates that schedule I generally produces the most accurate results on State routes carrying heavy traffic, and that schedule II gives the most accurate values on State routes that carry light traffic. Considering all stations, the weighted average deviation of the ratios of computed traffic to true traffic is approximately equal for schedule I and II, and schedule III is generally less accurate than either of the others. However, it may be remarked that the average differences are small under all three schedules.

A better comparison of the results may be made by arranging the number of stations under each schedule according to the percentage deviation of the computed traffic from the true traffic volumes, as indicated in table 9.

Traffic at 73 of the 99 stations may be estimated under schedule I within 5 percent of the true values, as compared with 74 stations and 54 stations for schedules II and III, respectively. While 14 stations give results within 1 percent of true values under schedule III, as compared with 14 under schedule I, and 18 under schedule II, results at 45 stations are more than 5 percent inaccurate under schedule III as compared with but 26 stations under schedule I and 25 under schedule II.

From these tests, at a limited number of stations well distributed both geographically and with respect to traffic volumes, it would appear that schedule III produces results with a considerably wider range of deviation from true values than schedules I or II; that is, the results from the use of schedule III are much more erratic than those from either of the other schedules.

Accuracy is one of the most important considerations involved in selecting a schedule of operation. Cost of operation, completeness of resulting data, and practical time and distance factors involved in putting the schedule into field operation are frequently of equal importance.

In the State-wide Highway Planning Surveys, volume is but one of the many traffic items investigated. At loadometer and pit-scale stations, weight of vehicle, weight of loads, length, height, and width of vehicles, origin and destination of vehicle trips are a few of the many additional items with respect to which information is needed. Classification of vehicles by types is also necessary.

At loadometer and pit-scale stations, flags, flares, and protection signs must be placed, since vehicles must be stopped for weighing and questionnaires must be filled out. This preparation of a station for safe operation takes a considerable amount of time. This time requirement, together with the time needed to transport from one station to another personnel trained to obtain this type of information, makes practically impossible the use of a schedule based upon short periods of observation.

Use of a short period of observation reduces the amount of effective time (i. e., percentage of total time that stations are actually in operation) and greatly in-

TABLE 8.—*Automatic traffic recorder averages for year 1939, local routes*

State	Station	Average daily traffic				Percentage of actual Average		
		Schedule			Actual average for year			
		I	II	III		I	II	III
Arkansas	10	250	266	268	[1] 259	96.5	102.7	103.5
Georgia	2	113	131	107	113	100.0	115.9	94.7
Iowa	609	96	93	107	96	100.0	96.9	111.5
	611	58	66	78	64	90.6	103.1	121.9
Kentucky	4	308	287	289	300	102.7	95.7	96 3
Maryland	8	341	344	363	349	97.7	98.6	104.0
Minnesota	169	130	134	122	[2] 136	95.6	98.5	89.7
	178	116	116	122	[3] 120	96.7	96.7	101.7
Montana	A-2	134	145	140	139	96.4	104 3	100.7
North Carolina	5	141	140	149	[3] 142	99.3	98.6	101.9
Ohio	5	155	176	160	172	90.1	102.3	93.0
South Dakota	105A	242	241	250	232	104.3	103.9	107.8
Texas	22	89	92	105	[4] 94	94.7	97.9	111.7
Wisconsin	19	186	185	196	195	95.4	94.9	100.5
Alabama	1	374	454	402	[5] 380	98.4	119.5	105.8
Massachusetts	3	209	225	175	[6] 213	98.1	105.6	82.2
	9	356	399	315	356	100.0	112.0	88.5
Michigan	683	335	325	311	330	101.5	98.5	94.2
Minnesota	177	547	533	526	567	97.5	94.0	92.8
	183	192	188	207	[7] 184	104.4	102.1	112.5
	184	229	202	203	[8] 199	115.1	101.5	102.0
Missouri	3	391	405	406	379	103.2	106.9	107.1
	4	440	491	431	470	93.6	104.4	91.7
North Carolina	6	213	231	182	[9] 213	100.0	108.5	85.4
	8	154	164	140	[10] 165	93.3	99.4	84.8
Ohio	3	241	257	242	[11] 261	92.3	98.5	92.7
	10	468	458	452	457	102.4	100.2	98.9
Oklahoma	10	562	558	549	558	100.7	100.0	98.4
Rhode Island	1	381	398	375	389	97.9	102.3	96.4
Texas	20	356	369	380	374	95.2	98.7	101.6
Utah	304	561	591	627	593	94.6	99.7	105.7
	307	1,500	1,660	1,585	1,593	94.2	104.2	99.5
Wisconsin	20	258	303	291	274	94.2	110.6	106.2
Weighted average						97.7	102.5	98 9
Weighted percentage of error						4.01	4.29	5.33

[1] Estimated.
[2] Aug. 6, 1938–Aug. 5, 1939.
[3] Aug. 20, 1938–Aug. 19, 1939.
[4] Nov. 19, 1938–Nov. 18, 1939.
[5] Apr. 30, 1939–Apr. 29, 1940.
[6] Jan. 15, 1939–Jan. 14, 1940.
[7] Jan. 29, 1938–Jan. 28, 1939.
[8] Mar. 26, 1938–Mar. 19, 1939.
[9] Sept. 11, 1938–Sept. 10, 1939.
[10] Feb. 13, 1938–Feb. 12, 1939.
[11] Year 1938.

TABLE 9.—*Number of stations at which computed traffic differs from actual traffic, under 3 assumed schedules; deviations by percentage groups*

Deviation of computed daily traffic from true daily traffic, percent	Number of stations		
	Schedule I	Schedule II	Schedule III
0–0.9	14	18	14
1–4.9	59	56	40
5–15	23	21	41
Over 15	3	4	4
Total	99	99	99

creases travel costs. Both these factors operate to increase very greatly the unit cost of an item of information, and thus the cost of the whole survey.

One advantage of either schedule I or II, as compared with schedule III, is that both provide much greater information with respect to normal maximum traffic volume. The maximum values recorded under either of the first two schedules are during periods of from 8 to 24 hours. Maximum values are ordinarily too irregular in their occurrence to permit an accurate measurement of them by means of a single hour of observation.

Still another consideration in the decision with respect to the most valuable schedule for field operation is the probable accuracy of the estimate of the proportions of the various types of vehicles—foreign vehicles, heavy trucks, busses, etc.—in the results obtained with various schedules. This question is difficult to investigate, partly because of the scarcity of data. To be sure, the automatic traffic recorder has now given a considerable sample in which is known the total number of vehicles during every hour of the year. However, it is clear that the number of foreign vehicles, for example, in proportion to total vehicles changes greatly throughout the year.

In summer foreign vehicles form 50 percent of the total traffic in some States. In the same areas in winter foreign vehicles are not over 15 percent of the total. In one State foreign vehicles are 14 percent of the total in December and 24 percent in August. The distinction between vehicle types cannot be made by the automatic traffic recorder, and detailed data classifying traffic throughout every day of a full year are available for but a small number of locations.

A limited amount of investigation of this problem at one station, considered to be typical of traffic found on most rural highways, is summarized in table 10.

TABLE 10.—*Classification of traffic by type of vehicle under various schedules*

Item	Passenger cars		Trucks and combinations	Busses	Total
	Local	Foreign			
Actual classification	78 2	7.1	14.5	0.2	100.0
Average of 8 runs (6 a. m.–10 p. m.) [1]	69.2	12.2	18.3	.3	100.0
Average of 8 runs (24 hour) [1]	69.2	11.9	18.7	.2	100.0
24-hour weekday, Saturday, and Sunday [2]	69.8	12.2	17.8	.2	100.0
24-hour weekday, Saturday, and Sunday [3]	84 6	3.2	11.9	.3	100.0
24-hour weekday, Saturday, and Sunday [4]	76.6	8.0	15.2	.2	100.0
16-hour (6 a. m.–10 p. m.) weekday, Saturday, and Sunday [4]	76.2	8 4	15.2	.2	100.0
8-hour (8 a. m.–4 p. m.) weekday, Saturday, and Sunday [4]	73.2	9.7	16.7	.4	100.0
Key station schedule (average of 5 runs)	78.0	6.8	15.0	.2	100.0
Average of 2 runs [5]	77.7	8.3	13.7	.3	100.0

[1] In months of probable maximum and probable minimum traffic.
[2] February and August.
[3] May and November.
[4] February, May, August, and November.
[5] 4-hour weekday, Saturday, and Sunday counts each season; staggered 8 a. m.–12 m. and 4 p. m.–8 p. m.

Other combinations, similar to those given in table 10, were examined and data for other stations were analyzed in the same manner. The tentative conclusion resulting from this analysis was that the standard key station schedule appears to give good results, but it is relatively a costly operation.

METHOD OF ESTIMATING TRAFFIC WITHIN CITIES OUTLINED

The above discussion includes an examination of the principal types of schedules that are, or have been, used in extensive traffic surveys on rural roads. Other schedules have been used in this work, but nearly all of them represent but minor modifications in the above general types.

Within cities, use has been made of a method of extremely short counts which was given practical application in a survey conducted in the city of Amarillo, Tex., by members of the Engineering and Police Departments in cooperation with the Texas Highway Department.[6]

Theoretically, under proper traffic conditions, a count of 1 minute during each half hour or hour might be sufficient for the estimate of total traffic, but the chief obstacle to this proposal was the loss of time involved by traveling between intersections. Finally, a 5-minute observation period was selected.

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[6] Traffic Aids to Texas Municipalities, by R. O. Swain. The American City, July 1940.

Time loss between stations was eliminated by stationing observers on the tops of the taller buildings in Amarillo. From certain of these buildings as many as 32 intersections could be observed. This procedure permitted a recorder to observe as many as six intersections within a half-hour period, counting traffic at each intersection for a 5-minute period.

The method used was described as follows:

> In estimating the hourly flow of traffic, the two 5-minute counts taken within a 1-hour period were added together and multiplied by 6. This method of short counts in towns and cities was determined to be as accurate as making full 8-hour counts and converting them into 24-hour figures. In checking the accuracy against the full count, the error averaged approximately 3 percent. * * * Intersections carrying more than 4,000 vehicles in a 12-hour period were within 3 percent of accuracy.[6]

Study of reports and tests now available indicates that: (1) The key station schedule, or a schedule of the same general type, produces a larger proportion of results within practical limits of accuracy than do the other schedules; (2) the 40-hour schedule (No. III) previously described produces results with a considerably wider range of deviation from true values at more stations than either the blanket count control or the key station schedule; (3) the blanket count schedule produces results comparable with those from the key station schedule; (4) collection of information such as that obtained at loadometer and pit-scale stations is a difficult matter from the standpoint of travel time and practical scheduling of field parties, and is uneconomical when based upon a short count schedule; (5) the short count schedule produces insufficient information with respect to maximum traffic periods; (6) the key station schedule produces accurate results in the classification of traffic by vehicle types; (7) the short count schedule by 5-minute periods produces results within the limits of practical accuracy and is useful in city traffic surveys, if the time loss and cost of travel can be reduced by stationing observers on tall buildings from which several intersections can be observed.

COMPOSITION OF TRAFFIC ANALYZED

Further analysis with respect to certain of these conclusions will be greatly facilitated by the accumulation of automatic traffic recorder data. Certain data are now available from vehicle classification counts taken throughout 1939 at 352 automatic traffic recorder stations located in 39 States. These data are of assistance in forming conclusions with respect to schedule selection.

The total traffic was separated by type of vehicle by means of classification counts taken at intervals throughout the year at the recorder sites. The number of vehicle classification counts in some States is small and, in some instances, it was necessary to supplement them by classification data obtained in years other than 1939. However, the proportions of the various types of vehicles change slowly from year to year, and the inaccuracy in the number of vehicles by type is slight.

Two hundred and ninety-four of these stations were located on the State highway systems, and 58 were located on local roads. An examination of the data discloses significant differences between the characteristics of traffic on these two classes of highway. A comparison of the results of the automatic traffic recorder operation with gasoline consumption indicates that the recorders furnish a measure of traffic representative of the country as a whole and, in States which are operating a large number of recorders, representative of traffic changes.

In two States the classification of vehicles was not as detailed as that reported by the other States, so that the discussion which follows applies only to the results of operation at 334 stations (276 on State routes, 58 on local roads) in 37 States.

The proportion of foreign traffic using State highways varies widely among the States, and is affected by two major influences: (1) The geographical location and size of the State; (2) the amount of recreational traffic that is attracted to the State as compared with the amount of local traffic. It is probable that in few States are the automatic traffic recorders sufficient in number so that, if manual operations were made at each location, representative averages of the amount of foreign travel would be obtained. In Florida, which attracts large numbers of tourists, foreign cars measured at 10 traffic recorders were nearly 40 percent of the total traffic. Nevada attracts a small amount of tourist travel, but, because of its geographic location adjacent to the Pacific Coast States, foreign travel at 11 recorders in Nevada was found to be nearly 40 percent of the total. Near the other extreme is Texas, in which foreign travel was slightly more than 10 percent, measured at 18 traffic recorders. Texas attracts a small amount of tourist traffic relative to its total traffic and is not crossed much by foreign vehicles en route to other States.

For all States, the percentage of foreign vehicles measured at automatic traffic recorders was 21.08 on State highways, and 1.72 on local routes, a ratio of more than 12 to 1.

Bus traffic was found to be less than 1 percent (0.88 percent) of traffic on State highways and negligible in amount upon local routes although, because of the low volume of travel on local routes, it amounted to 1.72 percent of the total. Busses are predominantly local vehicles; 14 out of 15 busses traveling State highways carry tags of the State in which they operate, and bus travel on local routes is almost entirely by local vehicles.

Heavy trucks (those with rated capacities of 5 tons, or more) use the highways but slightly more than do busses. They were found to be 1.01 percent of all vehicles measured at automatic traffic recorder stations, and nearly all were found on State highways. Eleven percent of heavy trucks counted were foreign vehicles as against 7 percent of the busses.

While the foregoing statement about the number of heavy trucks is true with regard to totals, an inspection of the detailed data discloses concentrations of heavy trucks much greater than those of busses. At several of the recorders located in California, Connecticut, Massachusetts, and Pennsylvania, heavy trucks averaged upwards of 100 per day during 1939 and reached 667 per day at stations 8 and 19 in Connecticut. At the single station for which data were available in Illinois, heavy trucks averaged 270 per day, while bus traffic at this station was but 13 per day.

A study of data for individual stations indicates a slight tendency toward increase in the proportion of heavy trucks with increase in volume of total traffic;

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[6] Traffic Aids to Texas Municipalities, by R. O. Swain. The American City, July 1940.

i. e., the percentage of heavy trucks tends to increase with an increase in the total number of vehicles using a route. In contrast, the percentage of foreign vehicles decreases generally with an increase in the total number of vehicles, although this tendency is not sharply marked.

From traffic counter records it is now possible to measure the seasonal variation in traffic volume during 1939 upon State and local routes, as indicated in table 11. Seasonal variation is similar on the two classes of routes, although the travel peak is earlier and higher on the State routes. The seasonal peak on State highways is in August, travel in that month exceeding that of the average month by nearly 24 percent. Travel on local routes is greatest in September and is about 17 percent greater than in the average month.

TABLE 11.—*Seasonal variation in total motor-vehicle traffic on State highways and local roads*

Month	Average daily traffic		Percentage of average month	
	State highways	Local routes	State highways	Local routes
January	1,608	276	75.68	80.00
February	1,607	245	75.63	71.01
March	1,838	278	86.50	80.58
April	2,018	311	94.98	90.14
May	2,165	335	101.89	97.10
June	2,306	358	108.53	103.77
July	2,594	394	122.09	114.20
August	2,633	396	123.93	114.79
September	2,384	403	112.20	116.82
October	2,233	390	105.09	113.04
November	2,104	384	99.02	111.30
December	2,007	370	94.46	107.25

SEASONAL VARIATIONS IN TRAFFIC FLOW COMPARED

The automatic traffic recorder data have been of invaluable assistance in the solution of another problem—that of estimating annual traffic volume when the period that traffic was observed covered but a few hours. There are hundreds of thousands of miles of public highways upon which traffic volume is below 25 vehicles per day, and only a limited expenditure for traffic information is justified upon such routes. At many intermediate points between key stations upon routes of considerable traffic importance, traffic need be observed only during short periods of time to produce acceptable data with regard to variation of traffic. At such points a factor derived from known traffic patterns (frequently from the continuous data collected at automatic recorders) can be applied in estimating annual traffic.

These factors must be based upon traffic patterns that are typical and reasonably invariant over a period of time; that is, they must be typical, or representative, in order that they will apply to many stations. They must be reasonably invariant because, if sharp changes occur in seasonal patterns (or other patterns needed), the factors derived for use in one year will not produce accurate estimates of annual traffic when applied to traffic data for short periods of time in later years. The term "reasonably invariant" is used because experience indicates that absolute invariance in patterns is not to be expected.

One measure of the invariance in seasonal traffic variation is presented in table 12, which shows seasonal variation of urban and rural traffic for each year from 1926 to 1931, inclusive, in Virginia. These figures are taken from graphs which accompany annual traffic flow maps prepared by the Virginia Highway Commission. Traffic data are available for the whole State highway system and are shown in the maps. The table indicates the remarkable lack of substantial change in the seasonal indices for both urban and rural traffic during these 6 years.

Other comparisons are shown in figures 19 to 23, inclusive, for Arkansas, Connecticut, Florida, Ohio, and Pennsylvania, respectively. In each of these States comparison has been made between the seasonal characteristics derived from former traffic surveys, with the seasonal characteristics of traffic at the automatic traffic recorder stations in each State operated during the year 1939. In each case the data are related to the average monthly traffic volume as 100 percent.

The number of traffic recorders operated in 1939 is much smaller than the number of stations from which the original seasonal indices were obtained. In the comparisons, data from States with the largest number of traffic recorders and an early traffic survey were used. Two of the States, Connecticut and Pennsylvania, each operated 22 traffic recorders during 1939; and in no State was the number of recorders less than 10.

TABLE 12.—*Seasonal variations in traffic on Virginia State highways* [1]

[Monthly variation in percentage of average monthly traffic]

Month	1926		1927		1928		1929		1930		1931	
	Urban	Rural	Urban	Rural	Urban	Rural	Urban	Rural	Urban	Rural	Urban	Rural
January	73	73	73	73	73	73	73	73	72	72	72	72
February	71	73	70	73	71	73	71	72	71	72	71	73
March	73	98	73	98	74	98	73	98	73	98	74	98
April	95	102	95	102	95	101	95	102	95	102	95	102
May	104	101	104	101	103	100	103	101	104	102	104	101
June	106	98	106	98	106	98	105	98	105	98	105	98
July	112	99	112	99	112	99	112	99	112	99	112	99
August	123	135	124	135	123	134	124	134	124	134	123	134
September	115	118	115	118	114	118	115	118	115	118	114	118
October	112	102	112	102	112	102	112	102	112	102	113	102
November	111	100	111	100	111	102	111	102	111	102	111	102
December	105	101	105	101	106	102	106	101	106	101	106	101

[1] Sections of highways within a 10-mile radius of cities are designated as urban, others as rural, by Virginia Highway Commission.

The Arkansas comparison, figure 19, shows a very slight change in seasonal variation from 1934–35 to 1939. In Connecticut, figure 20, the changes in seasonal variation are small. In Florida, the indices are also fairly close.

In Ohio the comparison of seasonal variation is between the years 1925 and 1939. Here the agreement is not so close as in the previous examples. And finally, in Pennsylvania, the comparison is for 1923–24 with 1939 and there is still greater disagreement between the indices than in Ohio. The data shown by these figures indicate that traffic volume has tended to be more evenly distributed throughout the year in the latter part of the last 15- to 17-year period.

In figure 23 the change from minimum to maximum values of the seasonal index in 1923–24 was from about 40 to 160, a ratio of 1 to 4. Corresponding values in 1939 are from a minimum of 65 to a maximum of 133, a ratio of 1 to 2.

Thus, while there has been a considerable change in seasonal indices over the longer period, with the increased reliability of operation of motor vehicles, better roads, and snow removal over the whole highway system, during the latter part of the period under discussion the apparent change in seasonal variation has been small.

TRAFFIC FLOW BETWEEN 7 A. M. AND 7 P. M. APPROXIMATELY 70 PERCENT OF DAILY TOTAL

It may also be noted that this relative invariance in seasonal change during the latter part of the period is more or less independent of the particular type of seasonal variation under consideration. For example, Florida's seasonal traffic indices differ widely from those of Arkansas and even more widely from those of Connecticut. The minimum traffic in Florida is in September; whereas in Arkansas, it is in January; and in Connecticut, it is in February. Nevertheless the change in seasonal variation is nearly the same for all three States.

There are other patterns of traffic that are reasonably invariant in the sense in which that term has previously been used. Tables 13, 14, and 15 contain data from automatic recorders on State routes, divided between heavy and light traffic routes, and on local routes, together with the computed ratios of several period totals to total daily traffic. An examination of these tables discloses interesting and significant facts. For example, the percentage of the total traffic moving between 7 a. m. and 7 p. m. is 71.9 for all routes and by classified routes is:

	Percent
15 State routes, heavy traffic	71. 0
21 State routes, light traffic	75. 8
14 Local routes	78. 1

R. O. Swain, in the article from The American City previously referred to, states:

> That hourly traffic flow also cuts certain patterns is another Cherniack theory which may be applied to Texas traffic. Of value in this connection is the movement of motor-vehicle traffic

Table 13.—*Traffic by hourly periods at automatic recorder stations, State routes, heavy traffic, 1939*

State	Station No.	Total yearly volume	Volume by time of day					Percentage of total volume				
			6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.	6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.
Massachusetts	1	1, 045, 290	357, 007	558, 993	129, 290	438, 746	720, 648	34. 2	53. 5	12. 3	42. 0	68. 9
	8	2, 110, 370	801, 639	1, 040, 069	268, 662	966, 815	1, 490. 998	38. 0	49. 3	12. 7	45. 8	70. 7
Connecticut	2	[1] 1, 365, 076	475, 784	624, 939	264, 353	590, 833	901, 065	34. 8	45. 8	19. 4	43. 3	66. 0
	17	[2] 2, 949, 154	1, 014, 094	1, 396, 555	538, 505	1, 194, 075	1, 913, 666	34. 4	47. 3	18. 3	40. 5	64. 9
Florida	10	1, 260, 823	548, 867	576, 310	135, 646	657, 113	961, 999	43. 5	45. 7	10. 8	52. 1	76. 3
	13	658, 659	274, 331	316, 150	68, 178	313, 335	493, 356	41. 6	48. 0	10. 4	47. 6	74. 9
Michigan	676	1, 265. 045	490, 464	623, 114	151, 467	596, 616	912, 106	38. 8	49. 2	12. 0	47. 2	72. 1
Louisiana	14	1, 109, 565	463, 276	516, 364	129, 925	543, 593	830, 583	41. 8	46. 5	11. 7	49. 0	74. 9
Missouri	9	1, 879, 116	704, 713	930, 552	243, 851	827, 618	1, 305, 289	37. 5	49. 5	13. 0	44. 0	69. 5
Texas	10	510, 302	228, 814	237, 471	44, 017	262, 527	401, 863	44. 9	46. 5	8. 6	51. 4	78. 7
Colorado	11	[3] 1, 944, 663	809, 980	924, 853	209, 830	945, 578	1, 452, 261	41. 6	47. 6	10. 8	48. 6	74. 7
Washington	10	1, 200, 884	495, 714	585, 353	119, 817	578, 385	892, 854	41. 3	48. 7	10. 0	48. 2	74. 3
Oregon	2	337, 721	148, 001	150. 424	39, 296	183, 120	256, 884	43. 8	44. 6	11. 6	54. 2	76. 1
California	5	289, 015	119, 347	142, 217	27, 451	153, 775	225, 398	41. 3	49. 2	9. 5	53. 2	78. 0
	10	1, 399, 962	538, 271	619, 621	242, 070	657, 227	960, 344	38. 4	44. 3	17. 3	46. 9	68. 6
Total		19, 325, 645	7, 470, 302	9, 242, 985	2, 612, 358	8, 909, 356	13, 719, 314	38. 7	47. 8	13. 5	46. 1	71. 0

[1] Feb. 18, 1938–Feb. 17, 1939. [2] Feb. 25, 1939–Feb. 24, 1940. [3] Dec. 17, 1938–Dec. 16, 1939.

Table 14.—*Traffic by hourly periods at automatic recorder stations, State routes, light traffic, 1939*

State	Station No.	Total yearly volume	Volume by time of day					Percentage of total volume				
			6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.	6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.
Arizona	5	72, 311	31, 060	35, 688	5, 563	38, 860	58, 239	43. 0	49. 3	7. 7	53. 7	80. 5
Arkansas	7	63, 629	30, 821	28. 739	4, 069	34, 725	51, 509	48. 4	45. 2	6. 4	54. 6	81. 0
Georgia	11	100, 559	42, 729	50, 124	7, 706	52, 483	77, 861	42. 5	49. 8	7. 7	52. 2	77. 4
Iowa	607	154, 537	65, 870	74. 135	14, 532	81, 364	118, 994	42. 6	48. 0	9. 4	52. 7	77. 0
Louisiana	13	[1] 54, 938	29, 262	22, 097	3, 579	31, 850	46, 445	53. 3	40. 2	6. 5	58. 0	84. 5
Minnesota	171	[2] 91, 365	36, 512	44, 943	9, 910	46, 443	66, 759	40. 0	49. 2	10. 8	50. 8	73. 1
	174	[3] 107, 561	52, 349	49, 145	6, 067	60, 716	85, 521	48. 7	45. 7	5. 6	56. 4	79. 5
Missouri	7	212, 287	91, 694	103, 239	17, 354	109, 796	165, 286	43. 2	48. 6	8. 2	51. 7	77. 9
Montana	A-7	173, 345	65, 897	89, 974	17, 474	85, 732	128, 910	38. 0	51. 9	10. 1	49. 5	74. 4
Nebraska	A-3	77, 735	32, 381	38, 198	7, 156	39, 365	59, 004	41. 7	49. 1	9. 2	50. 6	75. 9
Nevada	114	83, 728	35, 063	41, 519	7. 146	43, 385	65, 096	41. 9	49. 6	8. 5	51. 8	77. 7
New Hampshire	3	178, 359	78. 442	88, 170	11, 747	98, 509	141, 225	44. 0	49. 4	6. 6	55. 2	79. 2
Oklahoma	8	292, 276	118, 045	143, 160	31, 071	143. 739	217, 263	40. 4	49. 0	10. 6	49. 2	74. 3
Pennsylvania	7	[4] 130, 672	50, 721	68, 134	11, 817	58, 975	93, 890	38. 8	52. 1	9. 1	45. 1	71. 9
Rhode Island	3	118, 034	42, 640	63, 706	11, 688	54, 334	84, 617	36. 1	54. 0	9. 9	46. 0	71. 7
South Carolina	104	244. 745	101, 309	114, 861	28, 575	120, 267	179, 797	41. 4	46. 9	11. 7	49. 1	73. 5
Texas	8	287, 269	113, 812	140, 050	33, 407	136, 034	206, 308	39. 6	48. 8	11. 6	47. 4	71. 8
	9	180, 932	68, 411	95, 733	16, 788	81, 180	132, 950	37. 8	52. 9	9. 3	44. 9	73. 5
Utah	305	258, 738	111, 994	122, 549	24, 195	133, 345	199, 005	43. 3	47. 4	9. 3	51. 5	76. 9
Washington	9	71, 984	30, 552	35, 312	6, 120	38, 070	56, 209	42. 4	49. 1	8. 5	52. 9	78. 1
West Virginia	8	[5] 157. 409	69, 344	76, 527	11, 538	85, 479	125, 615	44. 1	48. 6	7. 3	54. 3	79. 8
Total		3, 112, 413	1, 298, 908	1, 526, 003	287, 502	1, 574, 651	2, 360, 503	41. 7	49. 0	9. 3	50. 6	75. 8

[1] Oct. 29. 1938–Oct. 28, 1939. [2] Aug. 6, 1938–Aug. 5, 1939. [3] Aug. 20, 1938–Aug. 19, 1939. [4] Oct. 1, 1938–Sept. 30, 1939. [5] Mar. 18, 1939–Mar. 17. 1940.

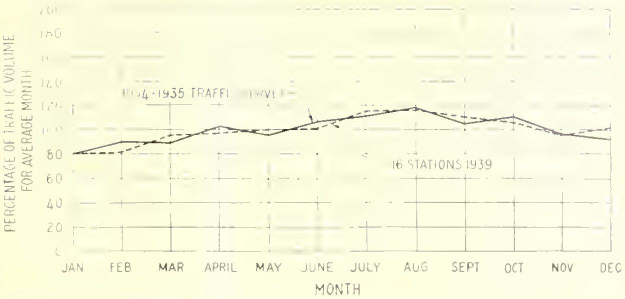

Figure 19.—Changes in Seasonal Variation of Traffic Flow in Arkansas.

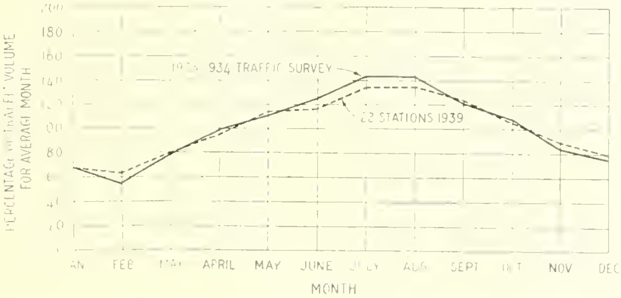

Figure 20.—Changes in Seasonal Variation of Traffic Flow in Connecticut.

between 7 a. m. and 7 p. m. Between these "daylight" hours, Cherniack figures show that approximately 70 percent of the traffic moves in both rural and urban areas. On Texas highways, according to data taken from the highway planning survey's 20 automatic traffic recorders, this "daylight" percentage is 73.23.

Thus the data shown in these tables agree with results obtained elsewhere. It is also significant that the proportion of traffic moving during daylight is greater on the local routes (78.1 percent) as compared with the proportion on heavily traveled State routes (71.0 percent).

Traffic during the period from 10 p. m. to 6 a. m. on the various classes of routes is 12.7 percent of the full 24-hour traffic, and is classified by routes as follows:

	Percent
State routes, heavy traffic	13.5
State routes, light traffic	9.3
Local routes	7.3

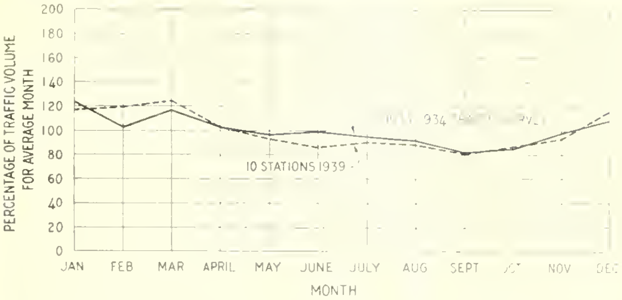

Figure 21.—Changes in Seasonal Variation of Traffic Flow in Florida.

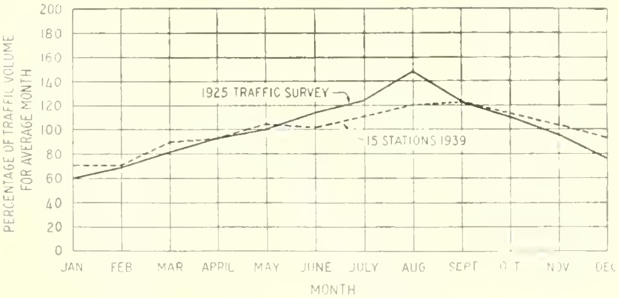

Figure 22.—Changes in Seasonal Variation of Traffic Flow in Ohio.

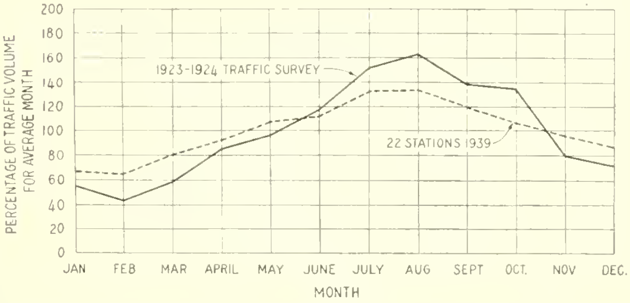

Figure 23.—Changes in Seasonal Variation of Traffic Flow in Pennsylvania.

Table 15.—*Traffic by hourly periods at automatic-recorder stations, local routes, 1939*

State	Station No.	Total yearly volume	Volume by time of day					Percentage of total volume				
			6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.	6 a. m. to 2 p. m.	2 p. m. to 10 p. m.	10 p. m. to 6 a. m.	8 a. m. to 4 p. m.	7 a. m. to 7 p. m.
Arkansas	10	74, 190	36, 487	33, 926	3, 777	40, 599	60, 699	49. 2	45. 7	5. 1	54. 7	81. 8
Georgia	2	41, 156	18, 965	19, 958	2, 233	21, 575	33, 702	46. 1	48. 5	5. 4	52. 4	81. 9
Iowa	609	34, 114	15, 051	16, 377	2, 686	18, 039	26, 314	44. 1	48. 0	7. 9	52. 9	77. 1
	611	23, 356	11, 810	10, 218	1, 328	14, 221	19, 586	50. 6	43. 7	5. 7	60. 9	83. 9
Kentucky	4	105. 911	48, 342	49, 544	8, 025	52, 402	82, 129	45. 6	46. 8	7. 6	49. 5	77. 5
Maryland	8	127, 563	58, 386	58, 588	10, 589	63, 817	95, 230	45. 8	45. 9	8. 3	50. 0	74. 7
Minnesota	169	[1] 49, 477	25, 646	19, 960	3, 871	27, 970	38, 409	51. 8	40. 4	7. 8	56. 5	77. 6
	178	[2] 43, 396	21, 171	19, 910	2, 315	24, 388	34, 898	48. 8	45. 9	5. 3	56. 2	80. 4
Montana	A-2	49, 019	19, 449	25, 801	3, 769	24, 976	37, 969	39. 7	52. 6	7. 7	51. 0	77. 5
North Carolina	5	[2] 51, 653	22, 429	26, 259	2, 965	27, 054	41, 559	43. 5	50. 8	5. 7	52. 4	80. 5
Ohio	5	47, 332	22, 701	22, 226	2, 405	27, 681	38, 701	48. 0	47. 0	5. 0	58. 5	81. 8
South Dakota	105A	86, 127	35, 792	42, 478	7, 857	43, 298	65, 770	41. 6	49. 3	9. 1	50. 3	76. 4
Texas	22	[3] 29, 887	13, 218	14, 692	1, 977	15, 530	23, 754	44. 2	49. 2	6. 6	52. 0	79. 5
Wisconsin	19	66, 841	27, 836	32, 600	6, 405	32, 843	49, 750	41. 6	48. 8	9. 6	49. 1	74. 4
Total		830, 022	377, 283	392, 537	60, 202	434, 393	648, 470	45. 4	47. 3	7. 3	52. 3	78. 1
Grand total, tables 13, 14, and 15		23, 268, 080	9, 146, 493	11, 161, 525	2, 960, 062	10, 918, 400	16, 728, 287	39. 3	48. 0	12. 7	46. 9	71. 9

[1] Aug. 6, 1938-Aug. 5, 1939.

[2] Aug. 20, 1938-Aug. 19, 1939.

[3] Nov. 19, 1938-Nov. 18, 1939.

Thus, the percentage of total traffic carried during the period from 10 p. m. to 6 a. m. for local routes is only half that for heavily traveled State routes.

The percentages of the total daily traffic shown in tables 13, 14, and 15 indicate that the 8 a. m. to 4 p. m. period is the best 8-hour period from the standpoint of uniformity of results for light-traffic routes, whether these routes be State highways or local routes. Records for 86 percent of the local routes vary less than 5 percent from the average during that 8-hour period, as compared with 81 percent for the light-traffic routes and 66 percent for the heavy-traffic routes on the State highway system.

These "reasonably invariant" ratios provide a measure of the accuracy of estimates of total yearly traffic volume made from traffic samples taken during relatively short periods of observation. The methods of deriving factors and their application have previously been discussed in PUBLIC ROADS.[3]

The results of automatic traffic recorder operations permit an analysis of the trends of traffic and, as the record accumulates, will be of increasing value for this purpose. As indicated in table 1, in 1937 there were 199 recorders in operation. However, not all of these were operated for the full year. While the record is now rather short, it may be stated that over this period the percentage increases in traffic at all stations closely approximate the increase in gasoline consumption.

It seems likely that the traffic data might provide a measure of business activity, both in general and for small areas or regions. The fact that both trucks and passenger cars are in the stream of traffic means that the data reflect business traffic as well as pleasure or recreational traffic. And since from 80 to 85 percent of all trips outside city limits are less than 20 miles in length,[8] local characteristics must be well represented in the data. These characteristics are essential in an index of regional business activity and, properly weighted, should combine to provide equally good indices of national business activity.

The chief value of knowing the trends of traffic is their usefulness in estimating future traffic. When it is recalled that many of the elements of the highway have a long life and that some of them, structures such as bridges for example, frequently require large expenditures, the importance of an estimate of future traffic is apparent.

The traffic estimate also provides a basis for estimating future highway income and thus permits the setting up of a rational budget for highways. The more accurate and representative the traffic trend, the more dependable and useful will be future plans of improvement. The automatic recorders furnish a volume of data covering a wide-spread area that are more accurate and more useful in trend analysis than any previously gathered.

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[3] Highway Traffic Analysis Methods and Results, by L. E. Peabody. PUBLIC ROADS, vol. 10, No. 1, March 1929.

[8] Preliminary Results of Road-Use Studies, by R. H. Paddock and R. P. Rodgers, PUBLIC ROADS, vol. 20, No. 3, May 1939.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF DECEMBER 31, 1940

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 4,224,798	$ 4,131,250	144.1	$ 19,864	$ 19,864	0.1				
Arizona	2,569,841	3,172,141	2,531,293	193.8	38,548	38,548					
Arkansas	3,352,061	3,366,371	3,333,924	363.4	18,137	18,137	.3				
California	7,747,928	8,293,122	7,747,928	262.7							
Colorado	3,395,263	3,300,163	3,182,248	113.7	194,968	194,968	.5				$ 18,047
Connecticut	1,418,709	1,570,314	1,394,591	22.7							24,118
Delaware	900,310	878,199	842,562	69.9							57,748
Florida	2,597,144	2,641,713	2,550,786	99.1	46,358	46,358	2.8				
Georgia	4,988,967	4,961,757	4,586,851	265.3	270,655	270,655	7.3	$ 66,224	$ 66,224	4.1	65,238
Idaho	2,222,747	2,346,077	2,222,747	187.4							
Illinois	8,694,009	9,073,672	8,694,009	518.5							
Indiana	4,941,255	5,635,125	4,941,255	239.2							
Iowa	4,991,664	5,356,873	4,989,104	528.6	3,899	2,560	.1				
Kansas	4,994,975	4,961,177	4,946,626	393.1	48,349	48,349	.4				
Kentucky	3,726,271	3,962,495	3,722,642	359.0							3,629
Louisiana	2,890,429	3,271,682	2,860,959	177.6	31,580	29,470					
Maine	1,676,799	1,707,665	1,676,799	77.6							
Maryland	1,750,738	1,822,945	1,696,484	44.8	52,000	46,859	.5				7,395
Massachusetts	3,262,885	4,249,700	3,184,283	19.2	49,328	49,321					29,281
Michigan	6,301,414	7,123,481	6,301,414	293.0							
Minnesota	5,277,145	6,516,457	5,272,746	901.6	4,399	4,399					
Mississippi	3,457,552	3,448,631	3,438,032	236.7				19,519	19,519	.3	
Missouri	6,012,652	6,142,947	6,012,652	777.5							
Montana	3,676,416	3,713,370	3,676,416	207.4							
Nebraska	3,870,739	3,962,855	3,826,572	374.6	31,896	31,896	.5				12,272
Nevada	2,243,074	2,423,114	2,243,074	110.2							
New Hampshire	945,225	987,794	940,765	42.0				4,568	4,460	.2	
New Jersey	3,129,805	3,337,889	3,073,505	35.7	46,947	46,947					9,353
New Mexico	2,871,397	2,878,564	2,869,041	213.6							2,356
New York	11,046,377	11,054,928	10,643,474	178.6	318,258	318,258	1.3	51,000	42,912	.6	41,733
North Carolina	4,720,173	4,800,532	4,720,173	289.2							
North Dakota	2,867,245	2,921,382	2,867,245	424.1							
Ohio	7,670,815	7,697,039	7,555,753	306.3	115,062	115,062					
Oklahoma	4,580,670	4,862,779	4,580,670	411.8							
Oregon	3,038,642	3,218,985	3,036,134	164.4							2,508
Pennsylvania	9,347,797	10,308,396	9,061,828	279.4	447,874	212,017	5.3	63,255	63,255	3.2	10,697
Rhode Island	989,208	1,113,526	989,208	18.8							
South Carolina	2,702,012	3,046,557	2,693,440	250.5	14,100	8,572					
South Dakota	2,976,454	2,987,929	2,976,454	513.8							
Tennessee	4,192,460	4,313,835	4,192,460	153.0							
Texas	11,989,350	13,141,777	11,989,350	1,126.6							
Utah	2,067,154	2,398,268	2,067,154	207.5							
Vermont	924,306	1,077,392	924,306	23.2							
Virginia	3,652,667	4,108,154	3,617,651	838.7	20,268	20,268					14,748
Washington	3,026,161	3,456,513	3,026,161	165.3							
West Virginia	2,231,412	2,535,594	2,197,783	100.5							33,629
Wisconsin	4,823,884	5,317,867	4,823,884	343.8							
Wyoming	2,219,155	2,227,966	2,219,155	155.0							
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	1,023,143	926,033	18.2							
TOTALS	195,000,000	207,893,653	192,948,370	13,249.5	1,772,490	1,522,508	19.1	204,566	196,370	8.4	332,752

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF DECEMBER 31, 1940

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 4,077,966	$ 4,013,492	54	2	12											$ 21,125
Arizona	1,256,099	1,303,815	1,217,704	15		7	$ 38,395	$ 38,395	2								
Arkansas	3,574,060	3,539,943	3,529,424	61	6	38	43,086	43,086									1,550
California	7,486,362	7,397,881	7,291,638	49	8	9	390,358	184,828	5								9,897
Colorado	2,631,567	2,643,408	2,569,252	30	3	27	61,208	61,208	1								1,107
Connecticut	1,712,684	1,593,553	1,573,492	9	2							$ 129,320	$ 128,444	1			10,748
Delaware	418,239	410,879	407,027	3													11,212
Florida	2,827,883	2,693,132	2,666,700	32	5	81	68,726	68,726	1								92,457
Georgia	4,895,949	3,852,003	3,582,662	64	16	60	120,144	120,144	1	2		111,356	111,356	2	2	4	1,081,787
Idaho	1,674,479	1,735,174	1,643,213	24	2	21						31,266	31,266	2			
Illinois	10,307,184	10,485,453	10,172,902	81	6		128,000	128,000		2							6,282
Indiana	5,111,096	5,383,293	4,997,797	42	15	163						116,200	113,298	1			
Iowa	5,600,679	5,787,906	5,588,962	108	9	8											11,717
Kansas	5,246,258	5,675,472	5,246,258	62	1	8											
Kentucky	3,672,387	3,754,416	3,537,407	27	13		190,024	134,281	1								700
Louisiana	3,213,467	3,086,514	3,049,366	30	3							158,498	158,498	3		14	5,603
Maine	1,426,861	1,435,159	1,416,643	20	4	4											10,218
Maryland	2,061,751	1,953,573	1,897,405	20	4	31	31,793	31,793		1							132,553
Massachusetts	4,210,833	4,012,883	3,987,617	28	4		145,395	138,962	1								84,254
Michigan	6,765,197	7,240,114	6,765,197	46	9												
Minnesota	5,395,441	5,483,227	5,394,329	86	15	50											1,112
Mississippi	3,241,475	3,216,129	3,205,792	62	7	28	4,780	4,780			2	7,460	7,460			3	23,443
Missouri	6,142,153	6,190,382	6,126,142	50	1												16,011
Montana	2,722,327	2,708,439	2,682,774	38	9		39,553	39,553	2								
Nebraska	3,556,441	3,428,924	3,369,502	85	3	23	186,939	186,939		1							
Nevada	887,260	912,949	887,260	9	3	3											
New Hampshire	822,484	840,475	803,800	10	4	7	18,684	18,684									
New Jersey	3,983,826	3,927,039	3,901,833	24	10	1	59,757	59,757									22,236
New Mexico	1,725,286	1,735,367	1,725,286	20	1	1											
New York	13,577,189	13,375,211	12,821,374	52	51		258,848	258,448		1							497,367
North Carolina	4,823,958	4,767,122	4,747,520	66	20	125						34,140	34,140			6	42,298
North Dakota	3,207,473	3,225,591	3,194,536	57	5												12,937
Ohio	8,439,897	8,617,125	7,915,169	51	8	14	126,598	126,598				388,062	357,000	1			41,130
Oklahoma	5,004,711	5,090,350	4,998,336	68	13	41											6,375
Oregon	2,334,204	2,392,554	2,324,175	17	7	2											10,029
Pennsylvania	11,483,613	11,372,784	10,666,619	86	22	9	486,261	486,261	3			115,846	115,846	1			214,887
Rhode Island	699,691	702,391	692,910	4	3		6,781	6,781									
South Carolina	3,059,956	3,002,428	2,893,217	49	19	47	22,381	22,381		1							144,358
South Dakota	3,249,086	3,278,998	3,249,086	76	5	76											
Tennessee	3,903,979	3,801,670	3,724,202	48	5	55	91,718	91,718		1		10,842	10,842			4	77,217
Texas	10,855,982	10,585,942	10,447,388	131	15	148	407,187	407,187	6								1,406
Utah	1,230,763	1,281,515	1,230,763	16	1	1											
Vermont	729,857	772,285	729,857	10	7	20											
Virginia	3,774,287	3,731,626	3,537,065	54	21	27	82,998	82,998	1			7,000	7,000			1	147,224
Washington	3,095,041	3,093,833	3,054,438	24	12	11	40,603	40,603									
West Virginia	2,677,937	2,669,262	2,595,213	28	4	7											82,724
Wisconsin	5,022,683	5,032,188	4,978,640	39	8	36	27,681	27,681			7	18,026	16,362			3	
Wyoming	1,360,841	1,364,049	1,356,655	15			4,186	4,186									
District of Columbia	410,804	432,570	410,804	3													
Hawaii	453,703	456,549	453,703	5													
TOTALS	196,000,000	195,551,511	189,272,546	2,088	391	1201	3,082,084	2,813,978	24	9	9	1,128,016	1,091,512	11	2	35	2,821,964

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF DECEMBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,858,664	$ 1,421,234	95.1	$ 5,150,759	$ 2,562,510	189.9	$ 994,020	$ 494,210	32.3	$ 1,767,202
Arizona	1,032,363	680,891	47.7	1,372,484	947,014	62.4	875,873	456,341	22.4	547,729
Arkansas	4,281,884	2,003,437	117.1	1,225,138	611,234	55.4	328,272	164,095	8.5	259,294
California	5,245,741	2,680,057	107.9	8,243,310	4,360,359	120.3	2,002,255	1,034,540	43.5	700,445
Colorado	2,009,290	1,103,155	186.8	2,142,320	1,226,351	82.6	547,420	308,526	85.0	1,721,244
Connecticut	1,696,143	833,513	15.2	1,268,769	608,618	11.3	159,030	77,409	1.0	1,026,887
Delaware	531,373	265,280	16.1	1,661,300	803,618	16.5				1,040,813
Florida	2,365,696	1,178,660	62.5	1,952,893	968,075	59.4	1,205,275	572,008	21.0	2,016,396
Georgia	2,950,486	1,467,507	182.1	7,094,248	3,547,624	275.2	1,813,878	906,939	84.6	4,759,094
Idaho	1,377,527	838,707	147.4	1,151,993	708,730	60.5	259,956	160,341	7.5	1,213,439
Illinois	5,317,670	2,629,572	137.1	7,893,692	3,946,481	141.2	2,698,300	1,349,150	82.9	1,393,069
Indiana	4,299,598	2,114,095	101.0	6,720,664	3,254,186	102.8	2,052,430	1,007,545	33.7	454,145
Iowa	5,204,137	2,421,733	177.1	4,136,513	1,862,459	130.9	808,508	357,016	35.6	151,439
Kansas	3,498,770	1,725,962	285.5	5,926,103	2,981,814	361.1	2,812,668	1,394,316	169.1	2,676,133
Kentucky	2,580,865	1,285,947	81.9	2,899,237	1,449,618	77.0	1,236,926	618,463	44.8	2,475,704
Louisiana	1,183,855	586,339	16.0	12,095,285	3,096,630	54.3	1,255,768	620,196	37.6	2,876,526
Maine	1,321,588	647,254	29.1	717,071	363,235	20.5	857,745	437,122	1.1	192,061
Maryland	828,000	411,500	19.5	3,312,558	1,650,361	32.0	507,303	253,651	5.9	1,183,923
Massachusetts	1,666,144	830,246	19.9	2,354,116	1,171,638	16.1				3,029,708
Michigan	5,989,548	2,837,520	210.1	8,061,710	4,018,255	192.3	321,110	160,555	23.0	336,090
Minnesota	5,518,764	2,700,527	435.9	4,708,910	2,351,071	256.3	1,237,733	618,249	35.7	2,617,570
Mississippi	1,841,191	741,378	94.4	7,282,834	3,396,411	353.6	1,015,400	487,050	77.8	1,071,737
Missouri	3,269,557	1,621,353	162.0	7,819,816	3,647,187	201.1	4,413,377	1,859,252	122.1	2,489,791
Montana	4,086,765	2,313,548	283.3	2,202,220	1,242,712	116.7	725,003	409,119	40.7	2,896,871
Nebraska	3,253,483	1,620,730	381.6	5,210,812	2,506,645	616.1	1,860,467	930,233	198.3	1,798,163
Nevada	1,399,393	1,201,043	74.6	1,182,681	1,030,030	48.9	196,013	170,882	16.3	587,190
New Hampshire	1,445,362	707,199	36.4	416,295	206,110	9.0	3,500	3,500		911,881
New Jersey	1,378,600	687,344	9.4	4,948,640	2,474,320	35.4	2,888,629	1,444,265	19.2	401,152
New Mexico	2,000,830	1,217,833	169.6	1,448,445	873,122	53.8	116,106	81,786	16.3	932,261
New York	10,542,515	5,179,059	182.3	11,999,783	5,967,340	154.3	620,127	273,309	10.1	211,796
North Carolina	3,874,928	1,936,027	200.3	4,898,892	2,433,660	216.0	690,840	344,925	31.9	1,060,354
North Dakota	1,849,256	994,635	179.5	2,611,958	1,457,832	209.1	2,566,896	1,312,736	217.0	3,139,076
Ohio	4,635,036	2,317,158	60.4	11,954,352	5,952,982	100.9	7,591,170	3,716,422	63.2	1,499,572
Oklahoma	2,228,276	1,181,616	109.9	2,867,349	1,493,571	77.9	1,688,011	861,309	73.9	3,558,298
Oregon	3,099,420	1,852,951	152.6	2,152,775	1,146,386	38.0	1,180,535	642,233	43.4	373,462
Pennsylvania	5,380,364	2,660,383	73.0	12,208,079	6,052,002	106.1	3,138,000	1,567,035	18.1	899,689
Rhode Island	1,006,582	502,165	10.2	1,213,296	605,822	10.9				767,776
South Carolina	1,329,331	640,444	95.5	2,005,435	963,966	148.0	2,055,790	859,666	66.1	1,605,701
South Dakota	3,147,009	1,767,729	533.0	3,813,803	2,411,163	459.0	837,400	483,150	129.4	1,985,853
Tennessee	2,291,094	1,137,140	55.7	3,155,900	1,577,950	109.7	1,504,412	752,206	46.6	3,107,302
Texas	7,317,938	3,596,505	463.9	8,325,052	4,114,232	366.2	4,292,219	2,125,070	180.3	3,577,768
Utah	983,647	714,222	73.1	891,740	667,400	39.7	543,279	293,840	13.4	559,107
Vermont	1,163,682	581,760	36.6	621,508	308,948	15.9	617,939	308,860	17.1	80,899
Virginia	1,985,813	908,195	56.8	4,069,045	1,929,063	78.2	565,028	282,514	5.6	986,199
Washington	2,887,210	1,494,860	70.4	2,711,360	1,447,035	37.0	613,531	328,500	4.8	363,856
West Virginia	1,956,930	974,878	74.2	2,834,444	1,411,021	57.6	1,379,066	687,410	22.8	943,029
Wisconsin	5,222,868	2,554,057	179.1	1,998,484	989,335	88.9	781,748	374,103	14.1	3,063,486
Wyoming	1,770,996	1,107,603	196.2	980,178	619,650	120.0	347,489	222,114	24.4	498,504
District of Columbia	550,699	275,349	5.0	147,418	65,675	.5	597,519	267,200	1.1	115,144
Hawaii	116,159	54,200	1.7	683,726	347,860	10.2	165,944	96,472	2.5	1,463,503
Puerto Rico	137,846	68,065	2.7	1,586,876	785,205	25.5	196,527	96,495	3.0	621,005
TOTALS	143,910,886	73,272,565	6,514.4	204,332,269	100,614,516	6,222.2	65,166,435	32,272,328	2,254.7	74,009,336

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF DECEMBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 190,944	$ 95,263	9.4	$ 1,303,357	$ 649,158	60.7	$ 5,000	$ 5,000		$ 276,238
Arizona	144,270	83,190	17.3	217,025	156,676	7.0				221,466
Arkansas	361,217	159,318	14.7	306,339	152,958	20.6	147,738	73,462	12.2	32,029
California	828,326	451,771	37.3	432,089	230,612	7.1	188,435	100,050	6.1	643,788
Colorado				298,599	167,773	6.9				122,450
Connecticut	370,531	179,413	4.6	105,456	49,907	1.8	126,629	56,207	2.6	109,035
Delaware	128,160	56,367	12.7	40,371	19,751	2.8				268,125
Florida	12,030	6,015		604,373	302,186	14.7	550,715	241,531	5.8	78,879
Georgia	118,120	58,526	15.6	777,663	373,831	63.3	363,156	181,578	37.0	1,000,215
Idaho	151,392	93,284	24.0	171,362	102,177	6.7	72,833	4,293	3.5	104,091
Illinois	1,605,055	791,594	80.3	1,038,750	504,375	28.5	271,500	131,550	16.5	104,114
Indiana	463,598	224,658	31.0	177,996	94,455	8.1	133,400	66,700	7.8	847,554
Iowa	2,193,253	1,039,631	464.3	689,523	328,000	187.0	284,278	123,275	86.2	56,021
Kansas	321,831	160,627	48.9	900,860	453,338	46.4	187,663	93,832	52.2	1,128,005
Kentucky	792,145	268,935	65.5	569,227	138,095	18.2	283,177	81,751	13.3	254,327
Louisiana	98,857	49,429	10.9	199,072	99,481	14.9				460,035
Maine	298,852	142,635	17.0	40,606	20,303	1.5				5,551
Maryland	94,300	47,150	3.5	132,390	66,195	6.5	36,000	18,000	4.3	385,314
Massachusetts	456,347	225,862	10.3	212,045	105,703	3.6				480,319
Michigan	1,447,559	715,572	112.3	347,480	173,740	29.1	590,520	288,230	41.2	292,245
Minnesota	641,778	315,634	86.8	742,976	371,488	116.8	290,629	145,314	37.7	765,584
Mississippi	172,962	86,481	10.6	717,352	353,176	39.0	366,200	160,615	14.3	435,223
Missouri	720,505	358,391	92.7	123,076	61,538	14.1	316,432	126,745	32.8	614,959
Montana	641,506	362,577	80.3	90,699	51,181	3.7	271,900	148,431	55.2	494,953
Nebraska	381,786	191,625	69.4	642,173	312,804	83.6	158,862	79,431	28.8	208,285
Nevada	201,288	172,834	40.9	175,048	152,379	14.3				8,573
New Hampshire	143,639	68,883	3.4	71,533	34,946	3.6				133,962
New Jersey	319,500	159,750	10.6	318,057	158,940	11,4				503,134
New Mexico	101,564	59,154	13.1	634,137	343,277	28.8	92,695	49,568	3.7	41,657
New York	1,698,625	805,985	52.5	1,654,255	827,127	55.4				77,918
North Carolina	941,132	468,665	82.2	225,403	115,313	25.5	151,390	75,695	14.6	219,706
North Dakota	42,880	24,583	.3	169,224	90,702	3.6				1,014,689
Ohio	1,263,398	631,633	46.5	2,199,455	1,098,468	67.0	160,375	80,187	9.9	677,442
Oklahoma	624,887	331,437	43.8	244,665	128,987	14.6	252,630	132,962	13.5	807,376
Oregon	372,237	205,770	56.4	180,645	74,674	13.4	192,677	98,260	18.2	161,630
Pennsylvania	1,418,725	701,839	51.0	940,699	469,441	19.2	224,936	112,468	4.7	147,408
Rhode Island	157,358	78,624	3.4	173,632	86,759	1.1				44,667
South Carolina	500,403	180,726	68.7	253,600	115,140	24.2	504,867	185,200	48.7	76,078
South Dakota				28,926	19,392	9.0				1,265,145
Tennessee	149,224	72,146	10.9	287,466	143,733	10.0				864,550
Texas	1,177,944	576,429	170.8	1,080,828	536,578	93.8	388,536	190,872	40.9	536,168
Utah	54,999	34,100	9.2	215,530	138,660	22.4	55,085	27,543	2.5	97,389
Vermont	346,351	112,834	13.1	196,176	57,330	7.6				4,391
Virginia	387,164	181,027	24.8	478,918	221,416	18.2	77,752	38,205	1.5	184,715
Washington	465,654	245,605	28.2	208,767	111,739	4.5	206,471	110,400	23.9	126,117
West Virginia	301,750	150,350	16.7	123,169	61,584	4.2	43,400	21,700	2.1	409,531
Wisconsin	330,613	164,243	7.4	636,776	318,310	23.2	369,473	141,704	16.1	496,435
Wyoming	429,615	253,344	42.8	153,530	95,100	9.1				70,846
District of Columbia	123,425	61,713	1.4	2,192	1,096					24,737
Hawaii	275,662	137,588	8.6	2,192	2,192					158,875
Puerto Rico	42,960	20,945	1.8	259,264	126,695	12.2	55,188	27,140	2.1	80,408
TOTALS	24,506,321	12,064,155	2,127.9	21,804,946	10,868,879	1,288.9	7,420,542	3,417,899	659.9	17,622,352

U. S. GOVERNMENT PRINTING OFFICE: 1941

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–20, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF DECEMBER 31, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 28,328	$ 28,328			4	$ 739,788	$ 719,711	7	1		$ 91,839	$ 91,839	1		5	$ 828,568
Arizona	192,342	184,976	3			179,037	178,688	1			19,260	19,260			4	232,120
Arkansas	558,358	558,213	5	1	7	775,385	771,164	8		1	302,508	302,453	3		8	173,007
California	439,428	439,428	4		6	1,028,139	847,540	7	1		757,323	757,323	2		14	599,979
Colorado						288,868	288,868	1			7,698	7,698			3	911,326
Connecticut	622,002	611,366	5	1		166,222	165,415	2								449,334
Delaware	68,080	68,080			13	126,685	126,685	5		3	2,332	2,332			1	467,605
Florida	207,524	203,025	2		2	102,816	102,291	1	1		139,739	139,739			25	1,209,730
Georgia	178,317	178,222	4	1	12	1,083,508	1,083,508	10	6		253,372	253,372	1	2	6	1,748,913
Idaho	239,542	236,113	5		16	34,107	34,107			6	43,471	43,471			15	401,036
Illinois	1,316,578	1,251,451	6	1	33	1,658,356	1,439,032	7	1	77	230,347	220,571	1		47	1,887,208
Indiana	578,286	578,286	3	2	50	629,262	629,262	4		40	392,007	359,107	1	1	11	661,505
Iowa	453,405	390,966	4		74	182,658	153,233	2		18	173,571	172,562	2		12	1,076,739
Kansas	687,637	687,530	9		14	303,791	303,313	3	1	4	535,580	535,580	11	1	10	727,701
Kentucky	573,093	572,549	8		11	637,876	637,876	7		1	382,102	343,702	2		4	240,666
Louisiana	95,496	95,496	1			345,122	291,627	2			632,768	575,037	11		2	811,244
Maine	159,759	158,841	1	1	6	132,646	132,646	1	1							263,934
Maryland	180,996	180,994	1		2	476,609	444,816	2	2	5	15,600	15,600			3	767,315
Massachuset's	15,710	15,710				343,592	333,170	1	1		90,040	89,740	1			1,928,300
Michigan	1,113,805	1,080,491	8	1	30	1,474,965	1,474,965	2	5	7	16,907	16,907			4	608,530
Minnesota	1,104,508	1,096,651	8	2	10	1,009,381	1,009,381	9	4	9	120,707	120,707		1	1	1,009,369
Mississippi	198,260	198,260	3			654,334	654,334	8	1		74,000	74,000	1		4	506,072
Missouri	1,208,389	1,208,389	6	5	1	1,709,501	1,254,081	5	1		222,144	137,078		2		901,314
Montana	427,675	427,675	5		2	88,047	88,046	1			9,155	9,155			3	344,634
Nebraska	230,657	228,986		1	9	993,774	993,774	15	1	1	143,457	143,457	2		12	158,591
Nevada	30,569	30,569			19	109,892	109,892	2			75,953	75,953	1		6	56,527
New Hampshire	100,989	100,953	3			149,458	148,638	3		1						331,169
New Jersey	269,185	269,185	2			596,798	596,798	3	1	4	264,880	264,880	1			932,729
New Mexico	242,979	242,979	2		6	183,821	175,247	3								479,380
New York	1,207,716	1,173,326	7	5		3,007,233	2,956,562	5	15		953,770	673,194	2	2	8	2,431,309
North Carolina	478,532	478,469	7	2	15	709,829	709,589	7	4	7	173,490	173,490		2	31	718,553
North Dakota	471,940	471,755	5		22	385,820	385,820	4		1						529,158
Ohio	811,713	789,951	6		16	2,143,380	2,084,190	9	3	1	865,625	850,163	5			2,065,990
Oklahoma	458,066	457,039	8	1	29	483,662	478,899	5		2	115,665	115,665			39	1,878,750
Oregon	208,639	117,537	3			254,208	197,981	1			86,974	80,790	2		2	290,984
Pennsylvania	1,387,269	1,377,793	13	2		2,021,009	2,017,181	14			872,546	872,546	7			2,621,372
Rhode Island	3,831	3,750				210,359	210,359		1							77,347
South Carolina	404,780	404,377	4	3	11	108,503	108,503	1		9	261,042	260,442	2		34	815,336
South Dakota	129,470	129,470	2	1	4	528,394	527,534	14			101,060	85,110	2			798,806
Tennessee	244,886	241,169	2	2		166,727	166,727		2		162,270	153,270	3	1		1,567,057
Texas	1,271,717	1,266,909	11		9	1,128,378	1,118,013	10	1		437,480	380,020	6		2	1,635,493
Utah	25,565	25,354			7	68,776	68,034	2		23	100,989	100,989			38	220,805
Vermont	101,024	101,024		1	4	148,111	148,111		1	5	8,182	8,182			3	213,187
Virginia	150,174	149,598	2	2	5	665,853	664,763	5	1	2	193,314	193,314	1	2	8	497,887
Washington	241,001	238,049	3	1	6	585,391	576,290	4	3		68,608	68,608		1	7	73,061
West Virginia	5,400	5,400			2	303,002	299,552	4			118,450	116,890	1		1	958,310
Wisconsin	834,332	819,761	6	4	7	452,980	423,971	3		4	127,215	117,651	1		14	1,262,972
Wyoming	1,987	1,982				560,905	560,904	6			4,997	4,997			2	166,634
District of Columbia	56,868	56,868	1			2,193	2,193									176,925
Hawaii	4,810	4,810			1	196,229	196,221	2			4,684	4,684			1	290,316
Puerto Rico						584,007	579,336	11								414,857
TOTALS	20,021,617	19,638,103	178	40	465	30,919,387	29,668,841	229	59	231	9,653,121	9,031,528	73	15	390	40,419,654

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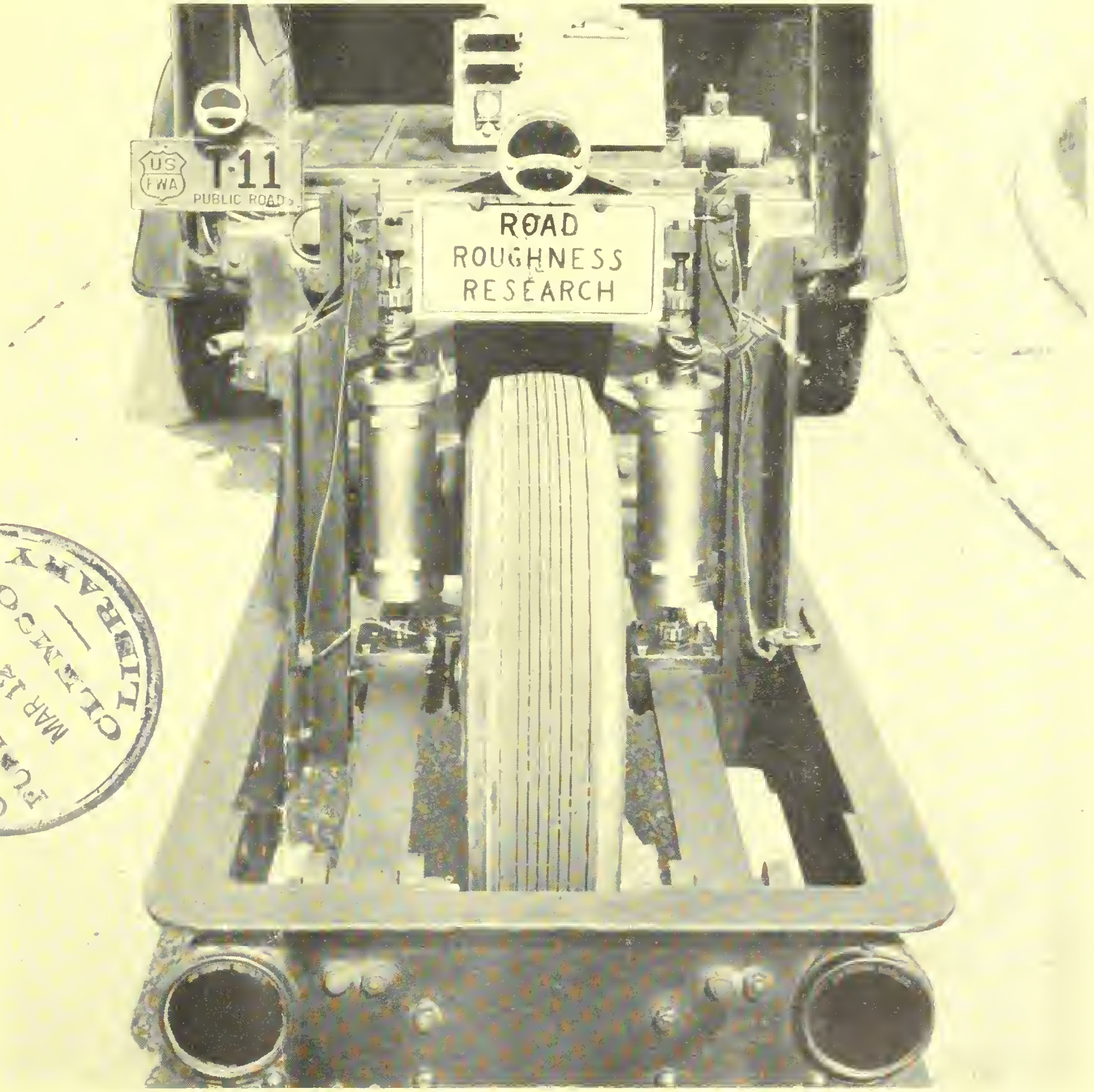
VOL. 21, NO. 12 FEBRUARY 1941



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PUBLIC ROADS

▸▸▸ *A Journal of Highway Research*

Issued by the

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 21, No. 12 February 1941

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

STANDARDIZABLE EQUIPMENT FOR EVALUATING ROAD SURFACE ROUGHNESS[a]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **JAMES A. BUCHANAN**, Engineer of Tests, and **ARTHUR L. CATUDAL**, Assistant Highway Engineer

Various methods have been employed for determining the smoothness or roughness of road surfaces. This report discusses these methods and certain inherent characteristics that have precluded their general acceptance. Equipment is described that operates on the principle of the measurement of the vertical movements of a wheel suspension with respect to its supported frame. In earlier applications of this principle it was recognized that the difficulties that were encountered were caused by the use of apparatus that made use of the motor vehicle and its load as a component part. This led to an investigation of the use of a special vehicle capable of standardization in all of its parts.

The new equipment is in the form of a single wheel semitrailer, attachable to any towing vehicle. The vertical movements of the wheel with respect to the frame are measured by a newly designed, overrunning clutch integrator with improved operating characteristics. The performance of the apparatus and the recommended standard operating procedure are given in detail. Many hundred miles of operation without mechanical failure indicate adequate mechanical design. The device is simple to operate and the data are obtained rapidly with it.

It is believed that the underlying principle as a means for indicating the relative roughness of road surfaces is sound and that the present equipment is superior to that developed earlier in two important respects: First, the use of a special vehicle has removed the uncertainties of vehicle operation that were always present when an automobile was a component part of the measuring apparatus; and second, the entire equipment is so designed that it can be exactly duplicated and, to this extent, the equipment is standardizable.

ONE ATTRIBUTE of the modern pavement in which the riding public is greatly interested is the smoothness of the surface. Because of this general public interest, great care is taken by highway engineers to construct and to maintain surfaces that are as smooth as possible, from the standpoint of riding quality. It is only natural, therefore, that those interested in the building and maintenance of highways should seek to develop tools with which to measure highway surface smoothness or roughness. These efforts began many years ago and have continued because none of the devices that have been developed has been completely satisfactory. Apparatus for indicating road surface roughness falls into one or the other of two general classes.

The first class comprises all of those instruments which produce, to some scale, a graphic profile along some element of the pavement surface. The earliest devices were in this category; and the forms of the apparatus and the operating procedures varied widely. In some instances, both in this country and abroad, the reference datum for the profile was determined by points fixed in the road surface. This procedure was very slow and tedious and led to the development of what might be termed a floating datum, one that was determined by a wheel suspension of some sort, 2, 4, 8, 16, and even 32 wheels having been employed in the various designs. Usually, the profile of the road surface, with respect to the datum, was traced on a ribbon of paper to some predetermined ratio of horizontal scales. These profilometers, as they were called, provided certain useful information as to the uniformity of the surface and the location of particularly rough areas. They were, however, awkward to maneuver, the data were difficult to analyze, and comparisons were tedious. Devices of this class were never very popular and their use has been quite generally abandoned.

VEHICLE SPRING DEFLECTION MEASURED AS INDEX OF ROAD SURFACE ROUGHNESS

The other class of instrument made use of the fact that irregularities of surface contour cause the wheels of motor vehicles to oscillate vertically with respect to the vehicle chassis. Since this motion is permitted by vehicle spring deflection, engineers conceived the idea of measuring the spring deflection as the vehicle traversed the pavement and of using the recorded deflection data as a measure of road surface roughness.

The manner in which the apparatus functioned varied rather widely. The recorded data were generally either a continuous graph of the vehicle spring deflection or a numerical integration of these movements in inches of deflection per mile of pavement length. Devices of this class have been given a number of different names, the descriptive but somewhat inaccurate title of "roughometer" having been frequently applied. As a class they have been rather popular because they were easy to use and comparative data could be obtained quickly with them; but also as a class they have had one important and fundamental deficiency which has long been recognized—the vehicle to which the device is applied becomes an essential part of the roughness indicating apparatus.

The Public Roads Administration has been interested in the problem of surface roughness measurement for many years and has experimented with devices of both classes. Various types of profilometers have been studied but, except for special uses on research projects such as small test tracks, there has been no recent development work on this class of instrument.[1] In 1926, there was published a description of one type of instrument of the second class.[2] Subsequently, the possibilities and characteristics of this particular instrument were studied by means of field and labora-

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[a] Paper presented at the Twentieth Annual Meeting of the Highway Research Board, December 1940.

[1] Two instruments of the profilometer type are described in Apparatus Used in Highway Research Projects in the United States. Bulletin of the National Research Council, vol. 6, part 4, No. 35, August 1923, pp. 14–16.

[2] An Instrument for the Measurement of Relative Road Roughness. PUBLIC ROADS, vol. 7, No. 7, Sept. 1926.

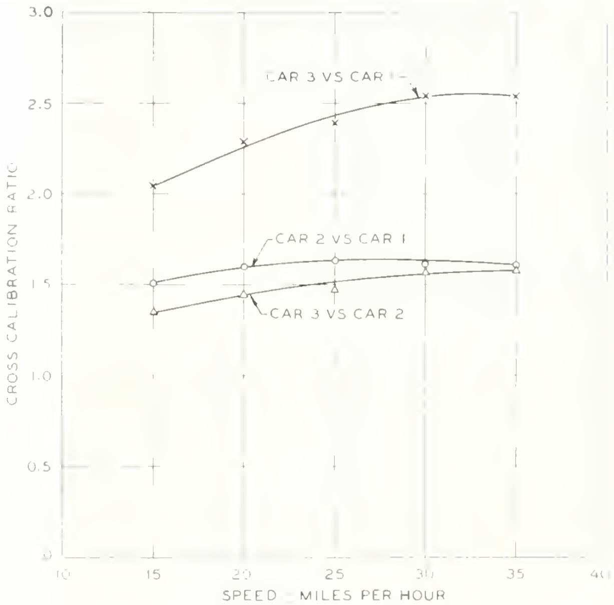

Figure 1.—Variation of Cross Calibration Ratio With Speed. (Each Value Is the Average of 4 Determinations.)

tory tests in which all of the various factors that affect its functioning were rather thoroughly explored. In this work particular attention was given to the intercomparison of the data obtained with various vehicles equipped with the integrator units and to the possibilities of calibration tests in which standardizable obstructions were placed upon a normal road surface and the spring deflections caused by these obstructions were recorded.

The tests were extensive and the data obtained were voluminous. However, it is neither practicable nor desirable in this report to describe in detail the work that was done. A few generalized comparisons will show the significance of the results of the study.

When two cars equipped with integrator units are driven over a unit length of a given road at a given speed, two values of integrated spring deflection are obtained. The relation between these two values may be expressed as a ratio. This ratio may change in value with car speed, and the manner in which it varies will depend upon the characteristics of the particular cars concerned. The variation of the ratio as determined for three cars and five car speeds over one road surface is shown in figure 1.

Not only does the ratio of the values obtained with two cars vary with the vehicle speed at which the particular comparison is made, but it varies also with the roughness of the road surface used for the comparison. This is illustrated by the data in figure 2. To obtain these data three cars closely spaced were driven on the same path over 15 successive miles of road at a constant speed of 25 miles per hour, integrator readings being recorded at the end of each mile. The variables present were the individual vehicle characteristics and the road roughness from mile to mile. It is apparent that these variables singly and in combination can cause rather wide fluctuations in the value of the cross calibration ratio. Other data bearing upon this point are shown in figure 3, in which the ratio was determined for two cars operated at a common and constant speed over roads representing a wide range of surface roughness. In this figure the road roughness values were those indicated by the instrument on car 3.

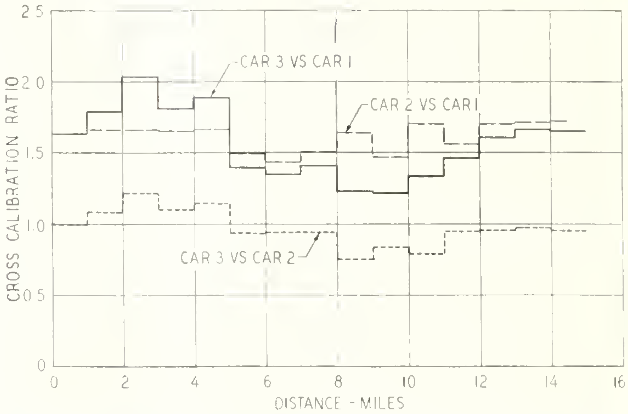

Figure 2.—Variation in Cross Calibration Ratio for Installations on Three Different Cars Operated at 25 Miles Per Hour Over Sections of Varied Roughness.

NEED FOR STANDARDIZABLE EQUIPMENT EMPHASIZED

Tests were made also with road surfaces whose natural roughness was increased by placing on the surface a series of artificial obstructions of fixed dimensions, shape, and spacing. These tests were made to determine the relation between the constant artificial roughness and the effect on the roughness values obtained for three different vehicles operated at the several speeds shown. The purpose of the procedure was to obtain values for the road surface roughness both with and without the artificial obstructions and, by subtraction, to obtain values attributable to the obstructions alone. The data shown in figure 4 are typical of those obtained. Each graph shows the range in the values of the increase in roughness created by a series of 10 obstructions placed 50 feet apart on each of three road surfaces and at five vehicle speeds. The three graphs show this relation for three different vehicles. The tests consistently showed that the indicated roughness caused by the artificial obstructions was affected by characteristics of the car and by the roughness of the road surface on which the obstructions were placed, as well as by the speed and other controllable test conditions.

The difficulties encountered were recognized as being caused by the use of apparatus of which the motor vehicle and its load are a component part and this recognition led to an investigation of the use of a special vehicle, capable of standardization in all of its parts.[3]

The investigation has gone forward and the development of a standardized vehicle revealed certain weaknesses in the integrator unit being used, with the result that an improved integrator has also been developed. The new equipment has been given extensive field performance tests and its characteristics have been carefully studied. It is with the design and performance of this new apparatus, susceptible of exact duplication in all of its parts, that this report is primarily concerned.

The design of the standardizable vehicle for evaluating road roughness is quite simple.[4] It consists of a rectangular frame within which is a single wheel equipped with a pneumatic tire. The axle of this wheel is attached to the center of two single leaf springs, one on each side of the wheel. The ends of the springs are attached to the front and rear cross members of the

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[3] The Dana Automatic Roughometer for Measuring Highway Roughness—discussion by L. W. Teller. Proceedings Highway Research Board, 12th Annual Meeting (1932).

[4] Plans and specifications for the apparatus will soon be available and will be sent upon request to anyone wishing to construct this type of roughness indicator.

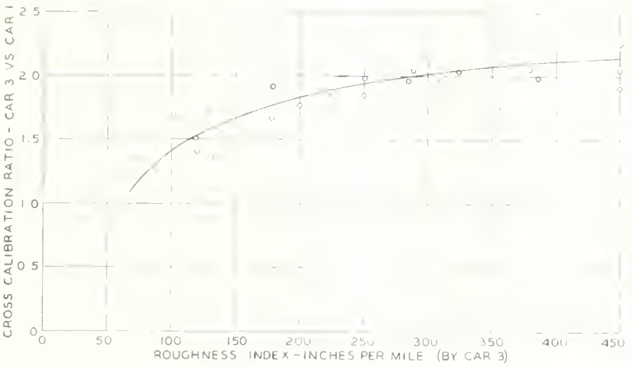

Figure 3.—Variation in Cross Calibration Ratio With Surface Roughness.

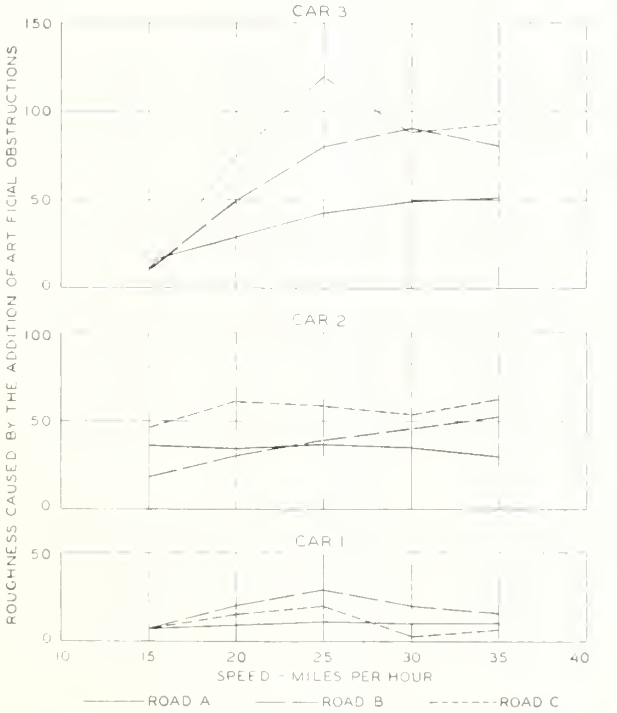

Figure 4.—Range of Increase in Roughness Caused by the Addition of the Same Artificial Obstructions to Three Different Roads Using Three Different Cars. (Each Value Is the Average of 4 Tests.)

rectangular frame through ball bearing fixtures. At the front of the frame is a tongue for connection with the towing vehicle. Over the wheel there is a cross-frame or bridge on which the integrator unit is mounted and to which the pistons of two dash pot spring damping devices are attached. The essential elements of this single wheel trailer unit are shown schematically in figure 5 and its general appearance, as seen from the side, in figure 6.

In the early consideration of the design it appeared reasonable to assume that the chassis or frame of this vehicle should be supported on standard tire equipment and that its sprung to unsprung weight ratio and its spring characteristics should, within reasonable limits, be proportioned in accordance with automotive practice. The gross weight of the entire apparatus as shown in figure 6 is 740 pounds, the reaction under the wheel being 580 pounds and that at the towing hitch being 160 pounds. It was decided that the chassis would be

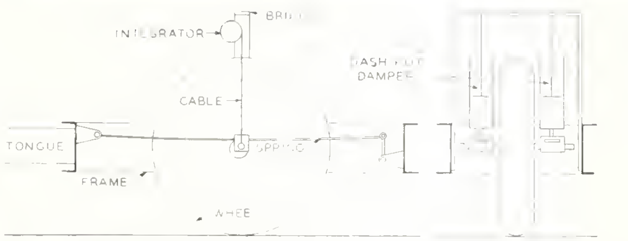

Figure 5.—Schematic Drawing of the Essential Elements of the Trailer.

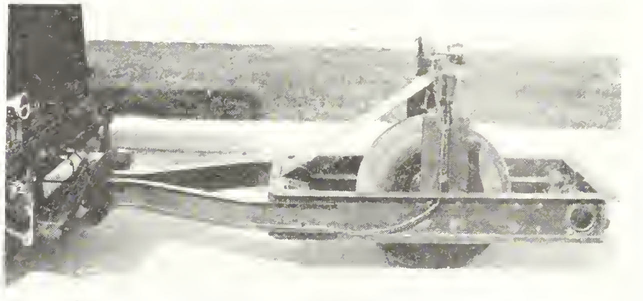

Figure 6.—Side View of the Trailer.

most convenient to handle if designed as a semitrailer that could be attached to any available vehicle. Finally, it was decided that a single wheel trailer would be more likely to repeat movements of its wheel with respect to its frame than would a 2-wheel trailer, because the former would respond to the contour of but one path.

A survey of tire equipment in use at the time of the design showed that the 5.50–17-inch balloon was the size in most widespread use on the then current models of automobiles. This size was selected since there was the greatest likelihood of its being available as a stock item of manufacture for many years in the future. However, during the development period it became evident that a tire of somewhat greater cross section would have certain advantages. A resurvey of possible sizes was made and the 6.00–16-inch 4-ply balloon tire was adopted. This is the size now in use and, as it still is one of the most popular, it probably will remain in production for many years.

The tire is mounted on a standard drop-center rim attached to a disk wheel. If the time should come when this disk wheel is no longer commercially available, it would be relatively simple to make one by welding a suitable flat disk to an unmounted rim. The hub is turned especially for this vehicle and is therefore exactly duplicable. The hub has a flange for the attachment of the wheel and is carried on the shaft by a pair of adjustable roller bearings. The axle or shaft is made of preheat-treated chrome molybdenum steel (S A E 4140).

DAMPING SYSTEM CONTROLS SPRING OSCILLATION

As mentioned previously, the frame of the trailer is supported on the axle by a pair of leaf springs placed longitudinally, one on each side of the wheel. These springs are pivoted at the forward end and shackled at the rear, as shown in figure 5. In the early work with roughness indicators installed on automobiles, one of the difficulties encountered was the lack of constancy in the spring action caused by variations in interleaf friction in the springs and in the friction of the spring shackles.

In designing the standardizable vehicle, therefore, it was decided to employ single leaf springs and to use ball bearing spring mountings. Standard grease-seal ball bearings of good quality, in specially designed fittings, are used.

The frame is constructed of standard steel channels and may, therefore, be exactly duplicated. It is rectangular in plan with heavy angle corner connections. The wheel is located centrally in the frame and over the wheel is the steel frame or bridge that carries the instruments and provides connections for the damping devices. At the forward end of the frame a pair of channel sections, forged to form a Y-shaped tongue, is attached to the frame. The towing connection is provided at the end of this tongue. A molded lead counterweight is secured to the forward end of the frame, its mass and location being such that the center of percussion of the entire trailer, when suspended from the towing hitch as a pendulum, is in the plane of the axle. The hitch to the towing vehicle maintains the trailer in an upright position but provides freedom of motion by means of a universal or gimbal joint device that can be attached to any towing vehicle.

Because of the absence of friction in the spring suspension, there naturally was a marked tendency for the spring-weight system of the trailer to oscillate in its own period when disturbed and some damping system to control this was necessary. It was further necessary that the damping system be consistent in its action under all conditions of temperature, that it be exactly reproducible, and that it be as simple as possible.

After some investigation and trial of existing devices available commercially, it was decided that none was likely to be satisfactory and that the damping requirements for this particular test vehicle would best be met by a simple dash pot arrangement in which an appreciable volume of low-viscosity liquid was displaced through fixed ports. The liquid could not be corrosive or subject to freezing, and its viscosity should be affected to the least degree possible by changes in temperature, evaporation, or chemical change such as oxidation. After extensive investigation, a satisfactory liquid was found, it being a mixture of a light mineral lubricating oil with a certain proportion of kerosene. The characteristics of this liquid will be discussed later.

During the development of the standardizable vehicle, the characteristics of the relative-roughness indicator[2] were being carefully studied also. It was found that improvement was desirable in four important particulars, which were—

1. The rack and pinion mechanism for translating the vertical reciprocating motion of the vehicle axle into the oscillating rotary motion that actuated the overrunning clutch was a source of error through lost motion.

2. There was a measurable amount of slippage in the three-ball clutch as designed.

3. The adjustment of the friction brake on the ball clutch was difficult to maintain and sometimes caused uncertain performance.

4. The mechanical counter required a rather long flexible shaft to connect the counter with the ball-clutch unit. Not only was such a mechanical connection poorly suited for use with the trailer because of its length and frictional resistance, but it was inconvenient as a connection between vehicles that required separation.

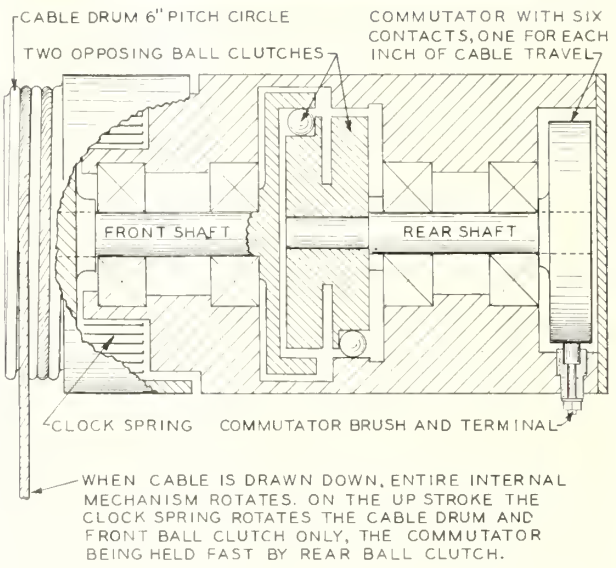
CABLE DRUM 6" PITCH CIRCLE
TWO OPPOSING BALL CLUTCHES
COMMUTATOR WITH SIX CONTACTS, ONE FOR EACH INCH OF CABLE TRAVEL
FRONT SHAFT
REAR SHAFT
CLOCK SPRING
COMMUTATOR BRUSH AND TERMINAL
WHEN CABLE IS DRAWN DOWN, ENTIRE INTERNAL MECHANISM ROTATES. ON THE UP STROKE THE CLOCK SPRING ROTATES THE CABLE DRUM AND FRONT BALL CLUTCH ONLY, THE COMMUTATOR BEING HELD FAST BY REAR BALL CLUTCH.

FIGURE 7.—SCHEMATIC SECTION OF INTEGRATOR ELEMENTS.

NEW CLUTCH DESIGNED FOR ROUGHNESS INDICATOR

In order to overcome the deficiencies just enumerated, a new clutch unit was designed. It consists of a drum and cable connection to the axle, a pair of opposed ball clutches, and a single brush and commutator for operating an electrical counter. This unit is shown schematically in figure 7.

The drum and cable arrangement was substituted for the rack and pinion in order to eliminate lost motion. The cable selected is of stainless steel, light and strong yet very flexible. It is 3-strand, 12 wires to the strand, each wire being 0.005 inch in diameter. It is rated at 175 pounds breaking strength, is a type used for deep-sea fishing, and is readily obtainable. The lower end is fastened to the axle with an adjustable connection. The upper end is wrapped around a spiral groove on a drum on the integrating unit which is supported by the chassis. The pitch circumference of the groove is 6.00 inches and within the drum is a clock spring that maintains a continuous and practically constant tension on the cable.

After careful study the principle of the ball clutch was retained but significant changes in the design were made. The early design had three balls in cylindrical tangential races, spring pressed against an annular inner race. The parts were relatively massive for the working radius of the clutch and it was somewhat difficult to obtain perfectly formed and polished races. The new clutch has a plain cylindrical outer race, while the inner race has eight steps that provide the wedge planes for eight spring-pressed steel balls. The outer race is 1.4 inches in diameter. The eight balls are ⅛ inch in diameter and they operate on a wedge angle of 15°.

In the earlier instrument a brake was provided to keep the driven part of the clutch element from following back the return movement of the driving part. This brake consisted of a wire loop held under tension against a grooved drum. The adjustment of the tension could be critical; if too loose follow-back would occur; if too tight clutch slippage would result. In the

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[2] An Instrument for Measuring Relative Road Roughness. PUBLIC ROADS, vol. 7, No. 7, Sept. 1926.

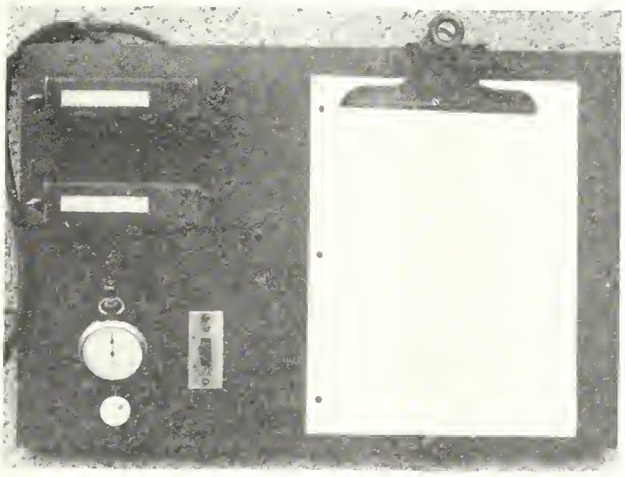

FIGURE 8.—CLOSE-UP VIEW OF THE INSTRUMENT BOARD.

new design the brake was abandoned and a second ball clutch identical in detail but reversed as to rotation was introduced, effectively locking the action against follow-back.

In order to record the progress of rotation of the driven element of the clutch and thus integrate the spring deflections that had occurred at any desired time, a commutator disk was built into the new instrument to operate electrically the remote recording unit. This eliminated the long drive shaft with its attendant disadvantages and made possible a simple wired connection between the integrating unit on the trailer and the recording instrument in the towing vehicle. The commutator is fitted with six equally spaced contacts and a single brush. Thus one revolution of the commutator causes six closures of the electrical circuit that actuates the counter. Since the pitch circle of the cable drum that drives the commutator is 6 inches in circumference, each impulse to the electrical counter marks the accumulation of 1 inch in the vertical movement of the axle with respect to the trailer frame.

The new instrument is entirely enclosed and sealed against dust and water. The internal parts are lubricated at assembly and screw-closed oil ports are provided for the introduction of specific quantities of lubricants during service, the procedure being to oil the clutch every 2,000 miles, allowing excess oil to drain out. Since most of the parts are lathe-turned, accurate alinement of the essential elements is insured. Both the driving and driven shafts rotate in ball bearings, the outer bearings in each case being provided with seals on their outer faces. The entire unit is rugged and compact, being about the size of a large drinking glass. It may be seen in the photograph taken from the rear of the trailer (cover illustration) at the right-hand end of the transverse bridge frame.

Completing the equipment is the instrument board carried in the towing vehicle. On it are mounted the counter that records the road roughness units, a second counter that records wheel revolutions of the trailer as a measure of distance traveled, a switch controlling both counters, and a stop watch. Since this instrument board provides a convenient place for data sheets, the operator usually holds the board in his lap and sits at any point of vantage in the towing vehicle. The appearance of the instrument board is shown in figure 8. The magnetic counters operate on 6 volts and the storage battery of the towing vehicle is a convenient source of power. Weatherproof extension wire and plug connectors are used throughout and a circuit breaker protects the system against overload.

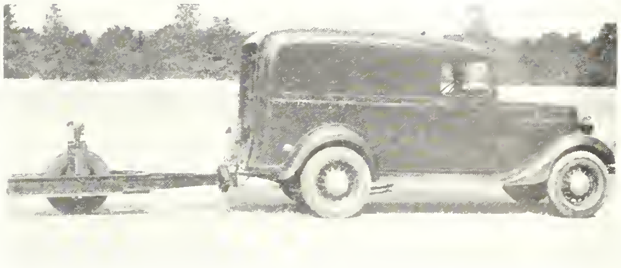

FIGURE 9.—GENERAL VIEW OF TRAILER AND TOWING VEHICLE.

STUDIES MADE OF VARIOUS OPERATING CHARACTERISTICS

The trailer is remarkably stable on the road at all speeds used in testing. It has good maneuverability, both in backing and turning. It is fitted with approved side and rear reflector plates and a combination stop and tail light controlled by the corresponding circuits in the towing vehicle. As a further safeguard, a red warning flag may be carried in a socket at the rear. A kit of emergency flares, lamps, and warning flags is carried in the towing vehicle. The entire equipment is shown in figure 9.

In the development of equipment such as that described, it is necessary to investigate the operating characteristics as affected by certain constructional details and by the conditions under which the equipment must operate in the field. These studies included investigation of the spring damping equipment, the vehicle springs, tire-equipment factors, the operating speed, effects of temperature, and the number of observations necessary to establish satisfactory data. As a result it is possible to specify the apparatus and technique that will give a high degree of consistency of performance.

The complete investigation of the influence of the tire equipment on the magnitude of the roughness values obtained included study of the effect of tread design, tread wear, and inflation pressure. As shown in figure 10, the first two were found to be not critical. Although almost any 6.00–16-inch, 4-ply tire will serve, it is believed preferable to use an all-rib tread design for the sake of uniformity and to replace the tire before the grooves entirely disappear through wear. A patched tire or one that has become otherwise unbalanced should not be used.

The inflation pressure, however, is important, as shown by the data in figure 11, and the technique adopted for testing requires an inflation pressure of 30 ± 0.5 pounds per square inch. It has been found by experience that after the trailer has been run for about the first 10 miles, some adjustment of the pressure is necessary but that during subsequent running little readjustment is ordinarily required. A calibrated pressure gage of the Bourdon tube type, with suitable fittings, is recommended for the measurement of air pressure as the ordinary commercial tire gages are not sufficiently reliable. A small air pump is carried in the towing vehicle.

As stated previously, it is essential that the damping equipment used on the vehicle springs be of a type such that temperature changes will have a negligible effect

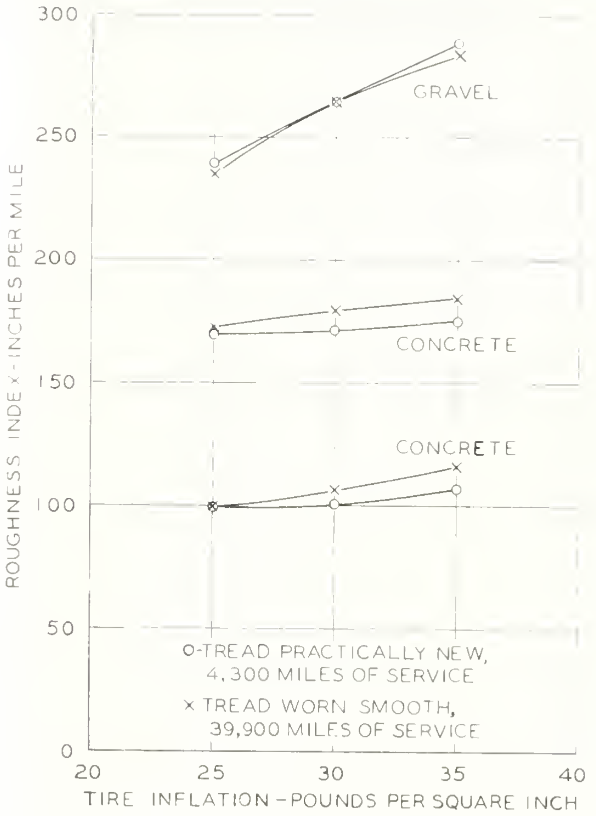

Figure 10.—Effect of Tread Wear on Roughness Index. (Each Value for Concrete Represents the Average of 12 Tests; for Gravel the Average of 6 Tests.)

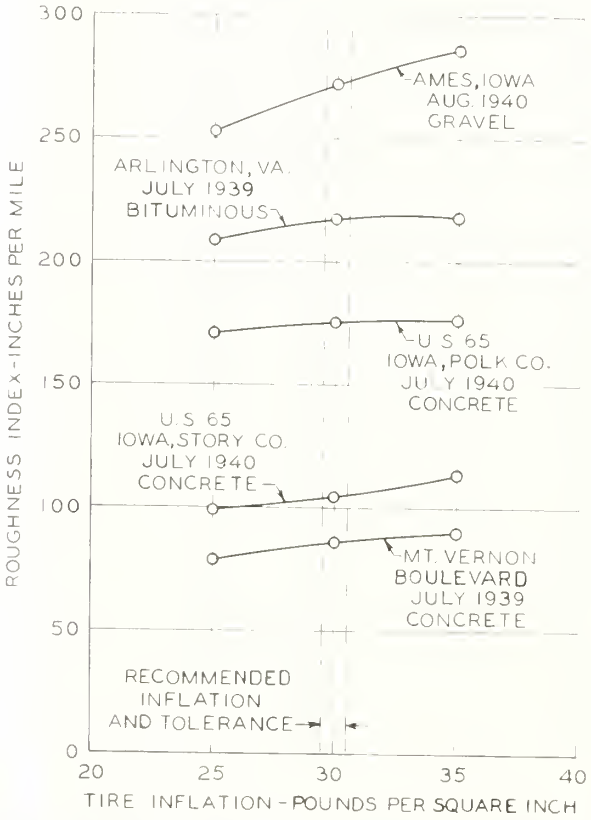

Figure 11.—Effect of Tire Inflation on Roughness Index; Nominal Speed 20 Miles Per Hour. (Each Value Is the Average of 6 Tests.)

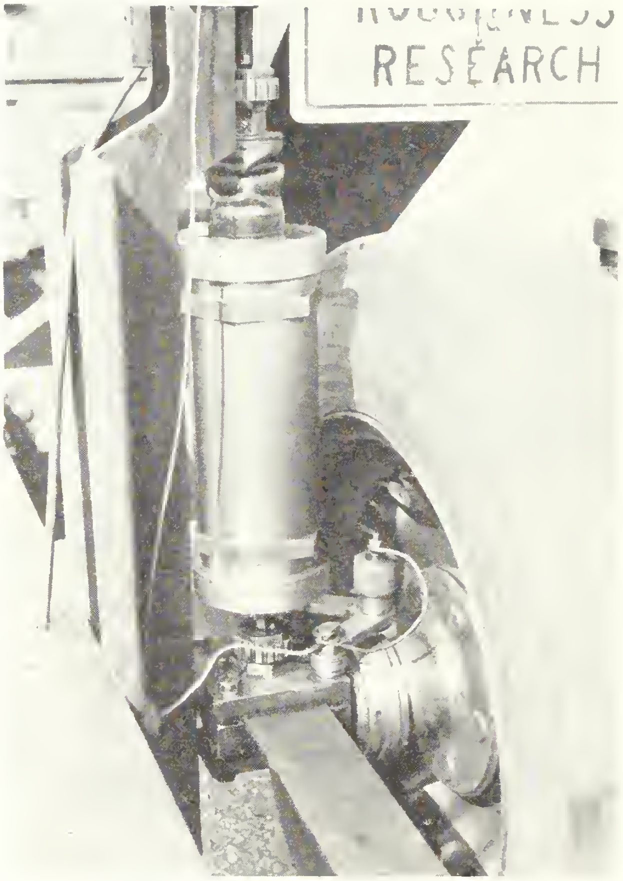

Figure 12.—A Close-Up View of One of the Damping Units and the Revolution Counter Contactor on the Wheel Hub.

on the damping action and also be designed so as to be exactly duplicable. The principle of a dash pot displacing a relatively large volume of low viscosity liquid was adopted because practical tolerances in the dimensions of pistons and ports and the viscosity changes due to the working temperature of the oil are less critical for such a design. As finally developed, the damper is a simple double-acting dash pot with a piston 3 inches in diameter. In the head of the piston are two bypass ports through which the oil flows. The ports are 0.25 inch in diameter and of specified length. Surge is effectively suppressed by ported baffle plates. The damper is connected to the trailer through ball thrust joints and each piston rod is protected against water and dirt by a collapsible leather sleeve. The close-up view shown in figure 12 shows the appearance of the unit.

The influence of oil viscosity on the integrator readings is shown by the data in figure 13. They were obtained from tests with three fluids, of which an S A E 20 lubricating oil was the heaviest. Attention is called to the wide range in roughnesses covered by the data. The study included many other fluids and from the data it was concluded that if one could be found that had a viscosity that would remain within the limits of 40 to 100 seconds (Saybolt Universal Viscosity) for the temperature range encountered in field testing, its use would prevent significant changes in the integrator readings when wide changes in air temperature occurred. The fluid must also meet other requirements as mentioned earlier. By trial it was

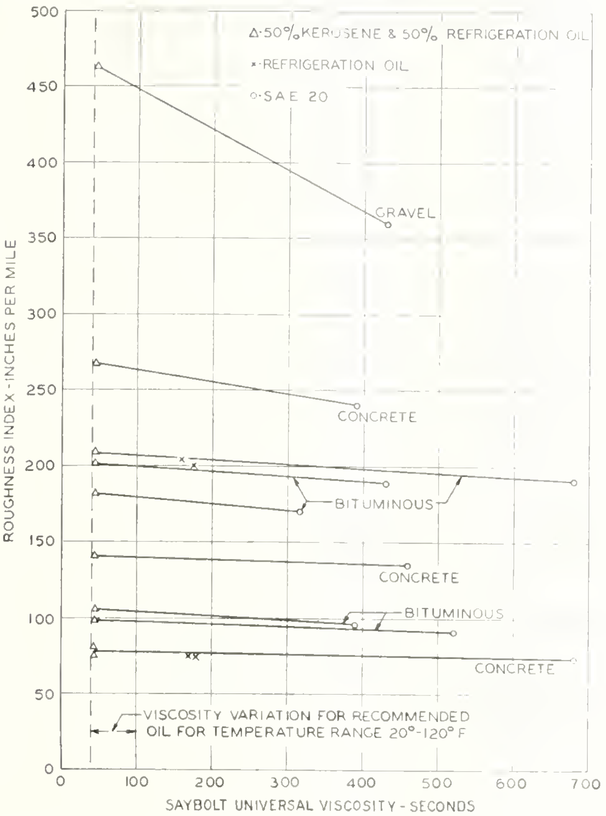

Figure 13.—Effect of Oil Viscosity on Roughness Index; Tire Inflation 30 Pounds Per Square Inch, Nominal Speed 20 Miles Per Hour. (Each Value Is the Average of 5 Tests.)

found that a light refrigeration oil mixed with an equal volume of kerosene satisfied the requirements. Figure 14 shows the relation between viscosity and temperature for the oil blend now being used and it will be noted that over the temperature range +20 to +120° F. the viscosity range is about 100 to 40 seconds (Saybolt Universal Viscosity).

During the development period various leaf springs were used, all obtained commercially with specifications that covered the dimensions and essential properties. It has been found that the commercial tolerances on width and thicknesses are sufficiently close for specifications for springs for this work. While some variation in the load-deflection rate is permissible, it is very desirable that the two springs selected for a given installation be closely matched for load deflection characteristics. The springs now in use are fabricated from silicon manganese spring stock 2½ inches wide by $^{11}/_{32}$ inch thick. The deflection rate is approximately 100 pounds per inch.

TESTING SPEED OF 20 MILES PER HOUR FOUND MOST SUITABLE

The speed at which the vehicle is operated affects the magnitude of the integrated spring deflections for a given road. Therefore, the speed at which the equipment is to be operated must be carefully selected and closely controlled. In selecting the speed for testing, the following conditions were considered:

1. The speed should be such that interference with or by traffic would be a minimum.
2. The speed should be within the range used by general traffic.

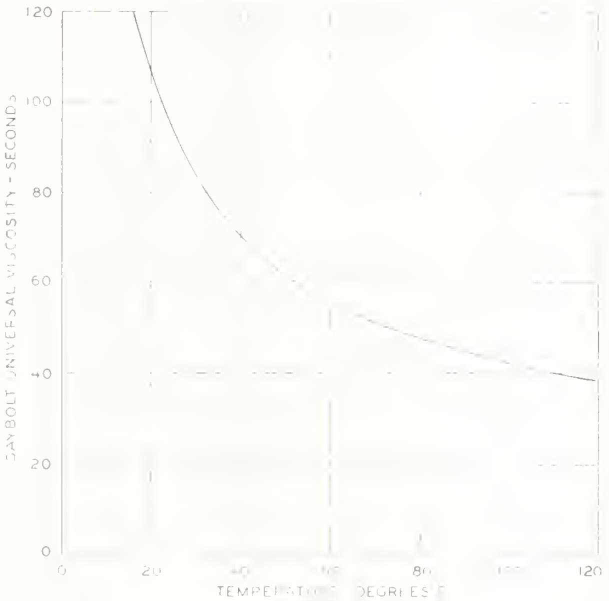

Figure 14.—Viscosity Curve for the 50-50 Blend of Kerosene and Refrigeration Oil Used in the Damping Units.

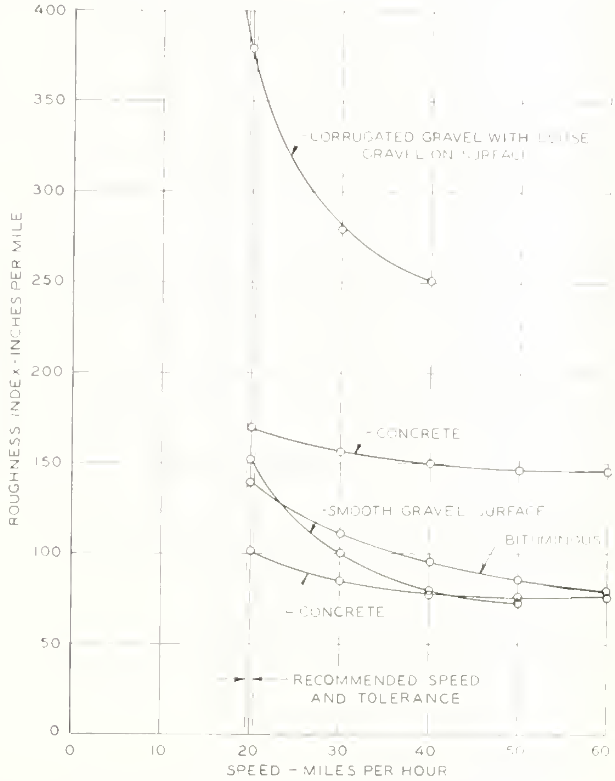

Figure 15.—Effect of Speed on Roughness Index; Tire Inflation 30 Pounds Per Square Inch.

3. The speed should be such that minor variations will not affect the integrator values to a material degree.
4. The speed should be slow enough to permit the operator to take adequate notes.

Although the trailer will function at speeds as high as 60 miles per hour, a much lower speed than this is necessary to meet some of the requirements stated above. On the basis of the experience and data thus far available a testing speed of 20 miles per hour appears

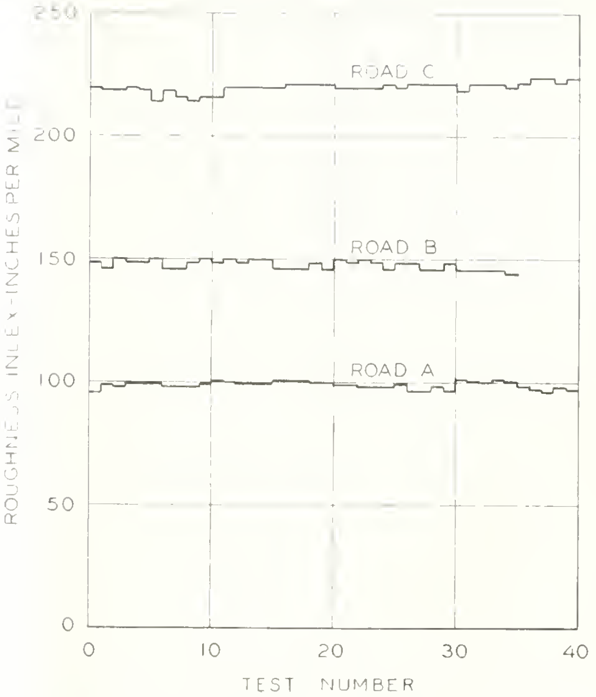

Figure 16.—Variation in Indices for Repeated Tests on Three Different Roads.

to meet these requirements best. In figure 15 are shown the relations between speed of the trailer and the magnitude of the roughness index for a number of typical road surfaces varying in type and degree of roughness.

According to these data, a variation in vehicle speed of ±½ mile per hour will not cause material variation in the roughness index at the 20-mile per hour speed. At this speed many miles of pavement can be tested in a day.

The vehicle speed and distances can be determined from the towing-vehicle speedometer indications but they can be obtained more precisely with the stop watch and revolution counter on the instrument board. A simple switch operated by a cam on the hub of the trailer wheel closes the circuit of the second magnetic counter once for each revolution of the wheel. From the counter, distances traveled can be easily determined and, in conjunction with elapsed time from the stop watch, average speeds can be quickly computed. Also this feature enables the operator to test any desired lengths of pavement without roadside markers, to calibrate the towing-vehicle speedometer, and to check the operation of the towing vehicle.

It may be noticed that the effect of operating speed, as determined with this standardizable vehicle and shown in figure 15, is somewhat different than the effect found with the earlier instrument on a conventional automobile (see fig. 1). This is to be expected since the spring-weight characteristics of the trailer differ from those of the automobile, the spring action is different, and the character of the damping provided in the trailer is quite unlike that in the automobile.

The influence of the characteristics of the towing vehicle on the action of the trailer was investigated by duplicate tests, using first a light passenger car with a 109-inch wheel base, and second a ½-ton panel body truck with a 113-inch wheel base. When attached to the passenger car the kingpin of the trailer was 33 inches back of the rear axle, while with the truck this distance was 64 inches. Test data obtained on six roads that covered quite a wide range in surface roughness are given in table 1. It is evident that there is no significant difference in the roughness indices for the various roads when the trailer was attached to these two vehicles. This is as would be expected from the dynamics of the trailer design.

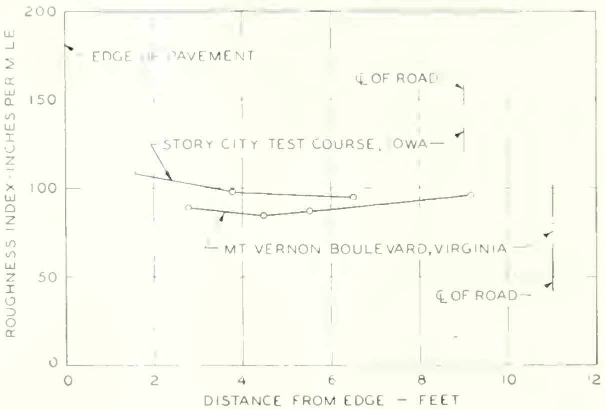

Figure 17.—Variation of Roughness Along Various Elements of Two Roads.

Table 1.—*Effect of type of towing vehicle on roughness index*

Type of road surface	Roughness index [1] when towing vehicle was—	
	Passenger car	Light truck
	In. per mile	*In. per mile*
Concrete	101	100
Bituminous	119	120
Concrete	159	160
Bituminous	214	216
Do	233	230
Concrete	289	289

[1] Each value is the average of 5 tests expressed to the nearest unit.

When operated according to the standardized technique that has been described, remarkable repeatability is obtained for a given track or path on a given road. In figure 16 are data obtained on three different roads in tests in which five measurements per day were made on a number of different days. For roads A and C the total period covered by the data in this figure is 8 working days but for road B the days on which the repeat measurements were made were spread over a period of about 4 months. In considering the matter of consistency, two facts should be kept in mind. First, no two runs would follow exactly the same path although the attempt was made to do so; and second, a dispersion of one point may be caused by the fact that the commutator segments of the integrator might happen to be either in or out of the circuit at the instants of closing and of opening the main control switch at the beginning and end of the test.

Different longitudinal tracks along a road may be different in roughness as is shown by figure 17. These data were obtained in tests that were carried out to explore this particular point. It is interesting to note that the apparatus functioned with a precision such that differences of this small order of magnitude could be detected. When appreciable differences in roughness exist across the pavement, a fair average should be obtained by making a number of tests along various longitudinal elements of the road surface.

(Continued on page 243)

SOME IMPORTANT CONSIDERATIONS IN THE DESIGN AND USE OF SOIL PRESSURE CELLS[a]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by A. C. BENKELMAN, Associate Research Specialist, and R. J. LANCASTER, Assistant Highway Engineer

THE DESIGN of most engineering structures must, of necessity, take into consideration the quality and the nature of the support offered by the soil on which the structure is to rest. Usually, the uniformity of the support is more important than the actual unit-supporting value, although both are of interest. Many and varied have been the theories proposed and the experiments performed in attempting to evaluate soil support; and, in order to provide experimental data, soil pressure measuring devices of numerous types, sizes, and characteristics have been designed and used.

For some time the Public Roads Administration has been studying the problem of pressure measurement in soils in connection with its possible application to the development of rational methods for designing nonrigid road surfaces. As a part of this study it is necessary to investigate the accuracy, consistency, and other characteristics of some of the devices that have been proposed and to determine the importance of some of the design features that affect the accuracy with which soil pressures can be measured.

Soil pressure cells have been used for many years in studying the problem of road subgrade support and pressure distribution. The results obtained in some cases have been such as to cast serious reflection upon the accuracy of the pressure-measuring devices used.

This report describes an exploratory series of tests that concern the performance characteristics of the pneumatic pressure cell, one of the more common types of soil pressure measuring devices. The tests serve to emphasize the fact that if a pressure cell is rigid and the soil in which it is placed is compressible, the accuracy of the pressure indication will depend upon the physical dimensions of the cell as well as upon the relation between the size of the pressure-sensitive area and the size of the total facial area exposed to pressure.

The investigation suggests the possibility of so designing the cell that the error in pressure intensity indication caused by cell thickness can be compensated for by suitably proportioning the active and inactive areas of the cell face.

This report describes certain preliminary or exploratory tests in which the attempt has been made to develop information concerning the factors that affect the performance of one type of pressure-measuring device, the pneumatic cell developed by the Public Roads Administration many years ago.[1]

That little importance has been attached to the external features of design of soil pressure cells is evident from even a casual survey of the literature on the subject. Apparently, the over-all size of such devices has been governed more by the space requirements of the internal pressure measuring element than by other considerations. Moreover, in some cells the pressure-receiving face is separate and distinct from the side walls and base section; in others, it is integral with these parts.

The data presented in table 1 indicate the extent to which the dimensional and structural features of some of the more recent designs for soil pressure cells vary. While the list is not complete, it serves to indicate that in the design of soil pressure cells no great consideration has been given to the possibility that the physical dimensions might seriously affect the accuracy with which they would indicate soil pressure.

EXTERNAL DESIGN FEATURES OF PRESSURE CELLS IMPORTANT

It is only reasonable to expect that the introduction into a soil mass of assumed homogeneity of a foreign object having radically different elastic properties will disturb the distribution of pressure in the vicinity of the object. It is, therefore, rather surprising that so little attention has been paid to this point in the design of pressure-measuring equipment.

Kögler and Scheidig[2] first called attention to the difficulties of measuring soil pressures accurately with a pressure cell. They pointed out that a cell more rigid than the soil would indicate pressures greater than those present in the soil and, conversely, a cell more compressible than the soil would give pressures less than those in the soil. This fact was also recently recognized by Goldbeck.[3] There can be little question as to the correctness of this reasoning and the natural inference is that if a device is to indicate true soil pressure, it must possess in itself the same elastic properties as those of the surrounding soil. In other words it must deform in all directions to the same extent as the soil.

TABLE 1.—*Variations in dimensions of various soil pressure cells*

Type of cell	Thickness	Diameter	Rim width (inches)
	Inches	*Inches*	
Pneumatic[1]	1.25	5.50	1.0.
Vibrating wire[2]	2.67	5.52	About ¼.
Carbon disks[3]	.25	.50	Thin bakelite ring.[4]
Rubber diaphragm[5]	.62	4.00	0.25.
Dynamometer[6]	8.50	8.00	Not indicated.

[1] A. T. Goldbeck, Measurement of Earth Pressure. Proceedings, Highway Research Board, vol. 18, 1938, part II.

[2] I. F. Morrison and W. E. Cornish, Measurement of Earth Pressure. Canadian Journal of Research, vol. 17, sec. A.

[3] M. G. Spangler, Wheel Load Stress Distribution Beneath Flexible Type Pavements. Proceedings Conference on Soil Mechanics and Its Application, Purdue University, July 1940.

[4] Thickness somewhat less than that of measuring element.

[5] F. Kögler and A. Scheidig, Druckverteilung im Baugrunde, Die Bautechnik, No. 29, July 1, 1927.

[6] W. H. Evans, Dynamometer for Measuring Earth Pressures. Engineering, vol. 149, No. 3876, April 1910.

With the possible exception of the rubber diaphragm cell, the cells listed in table 1 are essentially rigid in character. This means that the pressure indicated by them is apt to exceed the true pressure in the soil in which they are installed. The extent to which the indicated pressure might deviate from the true pressure probably would vary as some function of the thickness or of the cross-sectional area of the cell.

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[a] Paper presented at the Twentieth Annual Meeting of the Highway Research Board, December 1940.

[1] An Apparatus for Determining Soil Pressures, by A. T. Goldbeck and E. B. Smith. Proceedings, American Society for Testing Materials, 1916.

[2] Druckverteilung im Baugrunde, F. Kögler and A. Scheidig. Die Bautechnik, No. 31, July 15, 1927.

[3] Studies of Subgrade Pressures under Flexible Road Surfaces. Proceedings, Highway Research Board, vol. 19, 1939.

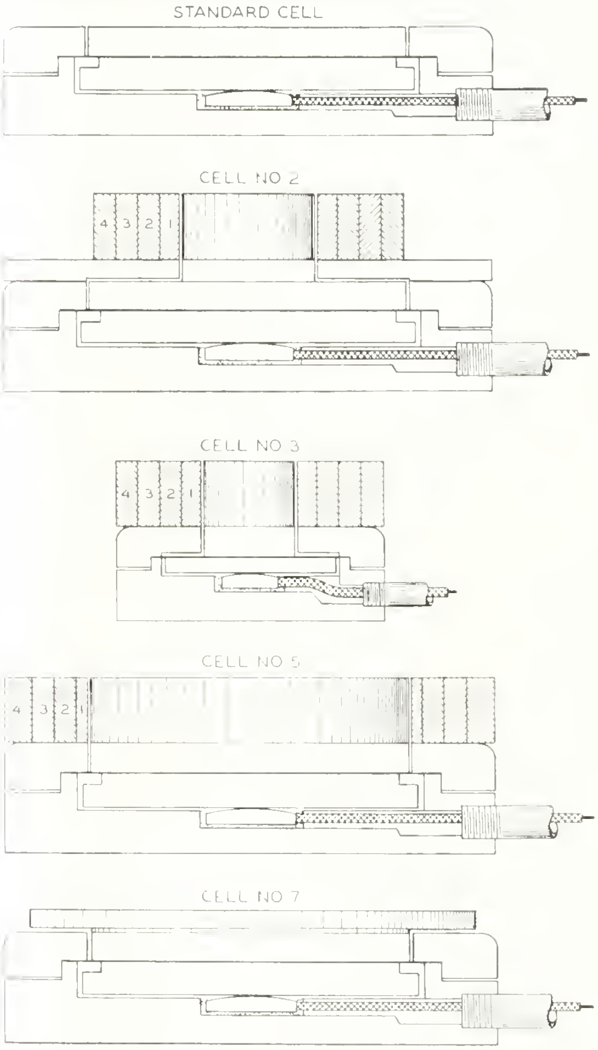

Figure 1.—Pneumatic Soil Pressure Cells and Attachable Sections.

In connection with this aspect of the problem, it is of interest to speculate upon just what sort of forces are imposed upon a rigid-type pressure cell when it is embedded in a compressible soil. It seems probable that they are analogous to those that resist the penetration of a rigid bearing block into the soil. It has been found that the pressure existing over the plane of contact between a bearing block and the supporting soil is not of uniform intensity. In the case of cohesive soils, the intensity is greater near the edges of the area of contact and in the case of granular soils it is greater in the interior portion of the area. This explains the observed fact that the resistance offered to the penetration of a circular bearing block into a cohesive soil varies inversely as the diameter of the area while, for a granular soil, it varies more or less directly as the diameter of the area.

Thus, if it is assumed that the forces imposed upon a pressure cell are essentially the same as those resisting the penetration of a body into the soil, it would be expected that the pressure indicated by cells of different size would vary with the area and would be different for cohesive and granular materials.

In this discussion it has been assumed that the cell is of tangible thickness and that the pressure-receiving face is not encompassed by a rim section. It seems reasonable that the presence of a rim around the pressure-responsive area would disturb the pressure-area relationships because it would tend to alter the distribution of pressure on the central area.

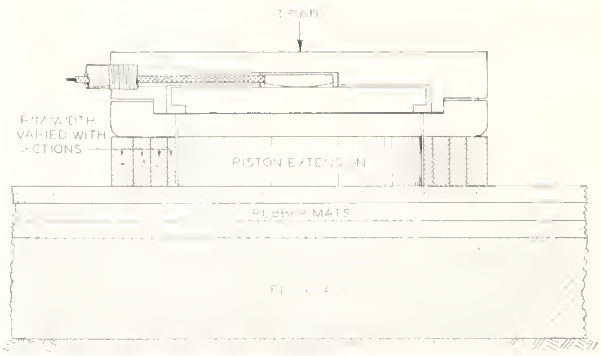

Figure 2.—Test Set-Up For Studying Effect of Width of Rim of Pressure Cell When Supported by Rubber Cushions.

Another point to be considered when cells with a rim around the pressure face are being used is that difficulty may be experienced in seating the cell upon the soil in such a manner that the same intimacy of contact exists over both the rim and the pressure-sensitve face. If uniform bearing on the two sections is not obtained, the indicated pressure might be considerably in error.

5 PNEUMATIC-TYPE PRESSURE CELLS STUDIED

In planning these exploratory tests, consideration was given to the several points mentioned in the above discussion. Pressure cells of the pneumatic type developed by the Public Roads Administration, but modified with the detachable rim and piston sections as shown in figure 1, were used in the study. Detachable pistons, not shown in figure 1, were also used on cell No. 7. In all cases the diameter of the pistons was the same as that of the pressure-sensitive area, referred to in the text and tables that follow. The important dimensional features of these cells are given in table 2.

Table 2.—*Dimensions of pneumatic-type soil pressure cells used in study*

Cell	Pressure-sensitive area		Cumulative width of attachable rim section			
	Diameter	Perimeter-area ratio	1	2	3	4
	Inches	*Inches*	*Inch*	*Inch*	*Inch*	*Inch*
Standard	3.57	1.12	------	------	------	------
No. 3	1.00	4.00	0.25	0.50	0.75	1.00
No. 2	1.50	2.67	.25	.50	.75	1.00
No. 5	3.57	1.12	.16	.41	.66	.97
No. 7	5.05	.79	------	------	------	------

It will be noted that the diameter of the pressure-sensitive areas varied from 1 to 5.05 inches and the perimeter-area ratios from 4.0 to 0.79. The construction made it possible to study the performance of these cells in a number of different ways. For example, they could be placed face down on a soil, or any other plastic material, and loaded; or they could be installed in a container that could be filled subsequently with a soil and loaded in any manner desired. In tests of the first type, the effect of variations in the width of the rim section was studied. In tests of the second type the cells proper were set flush with the base

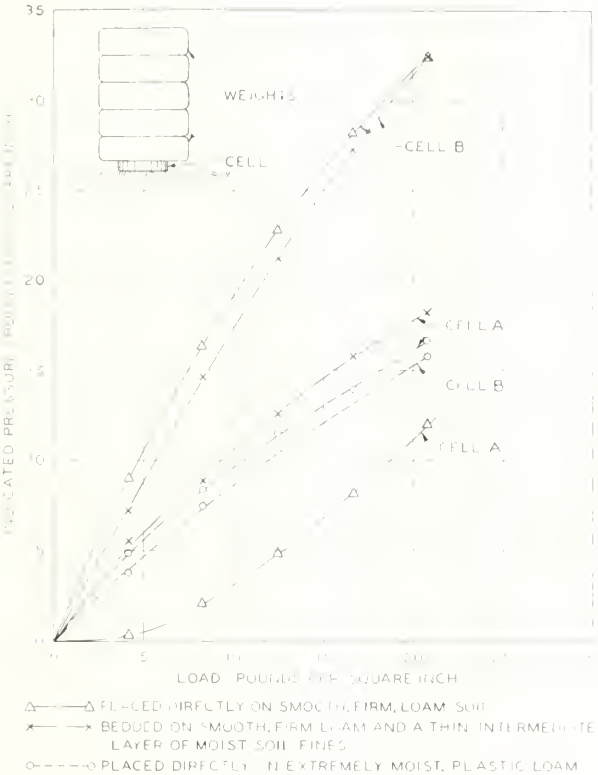

Figure 3.—Results of Pressure Cell Tests on Soil in the Field.

section of a cylindrically shaped soil container and, by means of the attachable parts, studies were made of the effect of thickness and facial area of the cells as well as of the effect of variations in the width of rim. In these latter tests the type and condition of the soil through which load was transmitted to the cells could be varied as desired.

The first series of tests concerned the performance of cells 2, 3, and 5 when placed face down upon rubber cushions and loaded individually as shown in figure 2. Two types of rubber mats were used: Ordinary soft sponge mats, ⅜ inch thick, and firm fabric-reinforced mats about ¼ inch thick. In the test a load was applied to each of the cells in suitable increments and the indicated pressures were noted, first with only the piston extensions bearing on the test materials, and second with the several rim extensions added one at a time.

The difference in performance of the cells when in contact with the two grades of supporting material was striking. In the case of the soft, yielding sponge rubber, the pressure intensity indicated by the cells was always equal to that computed for the load and the contact area in question. This was true regardless of the width of the rim around the pressure-recording piston. In other words, the pressure intensity over the contact plane remained uniform.

When the firm fabric mat material was employed an entirely different condition resulted. With this material difficulty was experienced in obtaining the same intimacy of contact beneath the central piston and the surrounding rim sections before the load was applied to the cells. This meant that in one instance more of the load was transmitted to the test material through the piston section; in another more through the rim section. This difficulty made it impossible to develop definite information regarding pressure distribution between such a material and the pressure cells when their size and rim width was changed. In contrast, the sponge rubber behaved much like a fluid or semifluid medium and slight differences in the relative elevation of the piston and rim section apparently did not affect the pressure intensity indications.

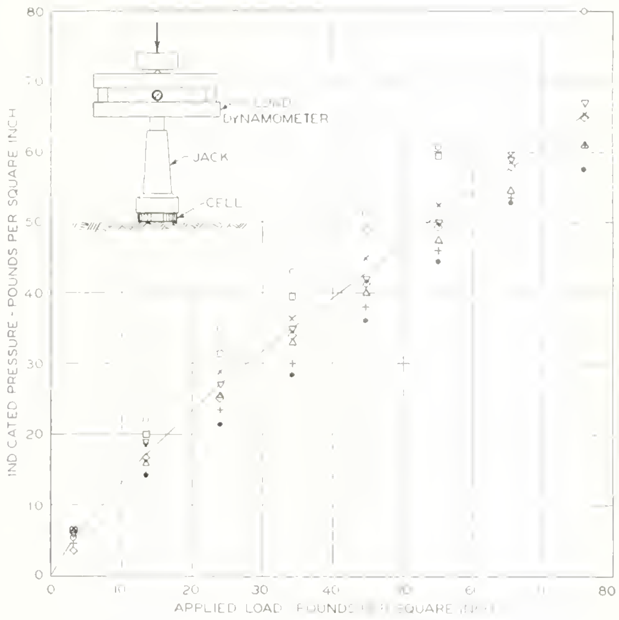

Figure 4.—Results of Pressure Cell Tests on Soil in the Field.

Another factor entering into such tests concerns the manner in which pressures are measured with the pneumatic type of cell. The piston or weighing face is subject to some outward movement when the cell is expanded and a pressure balance is obtained. If the material supporting the rim-piston face is strictly nonyielding in character, this movement, even though infinitely small, would tend to shift all the load to the weighing face. Goldbeck,[1] in a series of special tests made with damp sand soil, found that the error due to this movement was extremely small. However, it is believed that the magnitude of the error would be dependent upon the stiffness or deformation modulus of the material in question.

RIM-TYPE PRESSURE CELLS MUST BE CAREFULLY INSTALLED

Data showing the effect of variations in the seating on various soils of a cell, with its pressure rim face down, are shown in figures 3, 4, 5, and 6. The curves in figure 3 show pressures that were recorded in a test in which two of the standard cells (fig. 1) were placed face down in different ways upon different soils and loaded with dead weights as shown.

When the two cells were placed directly upon a smooth, flat surface of firm loam soil in the field, the relation between applied and indicated pressures varied widely. With an applied unit load of 20 pounds per square inch, the pressure intensity indicated by cell A was about 12 pounds per square inch and by cell B about 33 pounds per square inch. Apparently, in the case of cell A, a greater percentage of the applied load was transmitted through the rim section and less

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[1] An Apparatus for Determining Soil Pressures, by A. T. Goldbeck and E. B. Smith. Proceedings, American Society for Testing Materials, 1916.

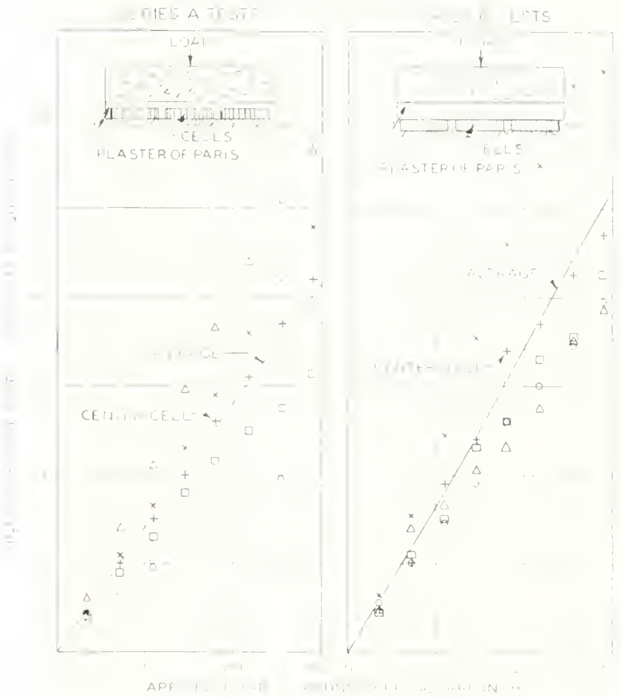

Figure 5.—Performance of Standard Cells When Placed In and Beneath Rigid Bearing Blocks.

through the pressure-sensitive area than in the case of cell B. In an attempt to improve the degree of uniformity of contact between the cells and soil, an intermediate thin layer of the moist soil fines was introduced and this resulted in some improvement, cell A indicating a pressure of 18 pounds per square inch (for a unit load of 20 pounds per square inch), cell B showing no appreciable change.

In another test, the same two cells were placed on an extremely moist, plastic loam soil, and the pressures indicated by the two cells were in close agreement although about 20 percent less than the calculated pressure intensity. In this test, it appears that there was a concentration of pressure near the perimeter or upon the rim section of the cells.

The data shown in figure 4 were obtained in another test designed to show the effect of variations in the seating of pressure cells of the rim type on a soil. Eight of the standard cells were placed reasonably close together on a thin bed of the moist soil fines laid over a uniform soil formation and loaded in the manner indicated. As the data in figure 4 show, the indicated pressures varied considerably between cells.

The data in figures 5 and 6 further emphasize the fact that when a group of pressure cells having rims are placed face down on a soil and loaded equally, there is apt to be considerable dispersion in the pressure values indicated by the various cells in the group. In the test referred to as series A in figure 5 four identical cells were spaced symmetrically around a central cell, then a bearing block 18.75 inches in diameter was cast centrally over the group, i. e., the cells proper became an integral part of the bearing block. Series B tests referred to in the same figure differed from series A in the one important respect that the cells were set into the soil rather than cast in the bearing block. The bearing blocks in each series were loaded in the manner illustrated. The data obtained in these tests likewise show an extremely wide dispersion in the pressure values indicated by the individual cells. The average indicated pressure for a given load is somewhat greater in series B than in the series A tests. This would indicate that the soil around and between the cells did not transmit its proportionate part of the load. This point was given further study in later tests.

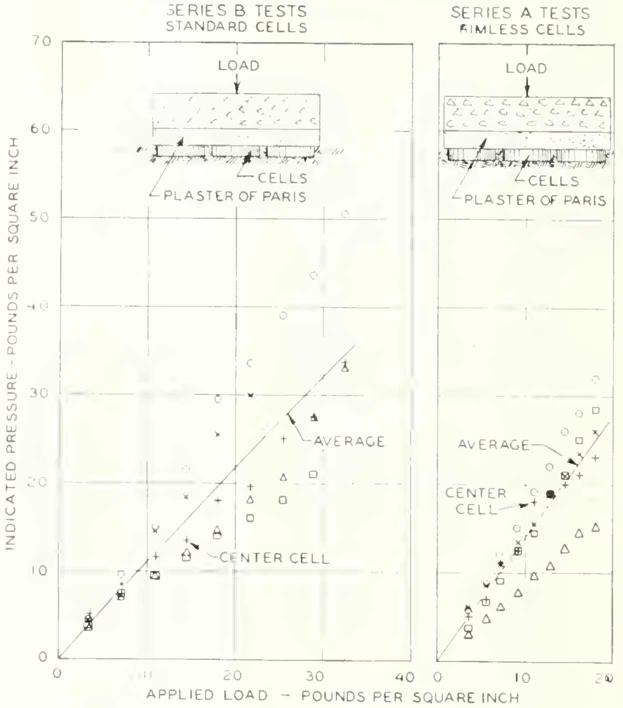

Figure 6.—Results Obtained by Testing Cells Beneath Rigid Bearing Blocks.

The possibility that much of the dispersion in pressure values indicated by different cells when tested in the same manner was caused by lack of uniformity in seating them on the soil was investigated in another group of tests, the results of which are shown in figure 6. In the series A tests referred to in this figure, cells with the rims eliminated (cell 7, fig. 1) were installed beneath the bearing block. It is to be expected that there should be less difficulty in properly seating cells without rims and, in general, the data indicate this to be the case. With the exception of data from one of the outer cells, the values for the various cells are reasonably close together, in contrast to those of series B, in which the standard rim-type cells were again used, and in which the individual pressure values are scattered to about the same degree as in the earlier tests. The fact that the average pressure indicated by the cells in the series A tests is higher than that in series B is probably due to the greater cell thickness. Other tests were made to investigate this matter more fully.

RIGID PRESSURE CELLS MAY DISTURB THE CONTINUITY OF STRESSES

The tests thus far described clearly indicate that considerable trouble may be expected in so seating a pressure cell of the rim type on a soil that the same initial intimacy of contact obtains beneath the rim and pressure-recording area. This appears to be true even though the cell is bedded on a thin intermediate layer of moist soil fines. Thus, where it is necessary to place the cell on a soil such as, for example, a prepared section of road subgrade, rather than place the soil upon the cell as in a fill, the presence of a rim might cause ap-

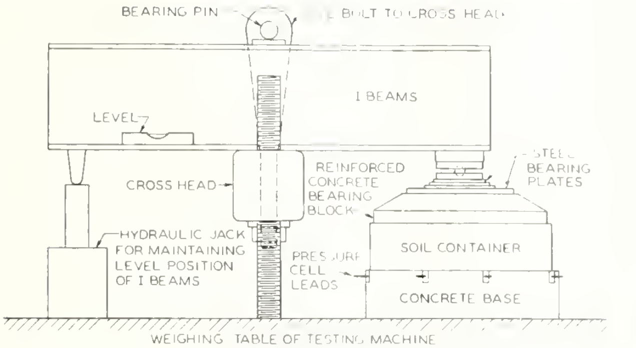
BEARING PIN
BOLT TO CROSS HEAD
I BEAMS
LEVEL
CROSS HEAD
REINFORCED CONCRETE BEARING BLOCK
STEEL BEARING PLATES
HYDRAULIC JACK FOR MAINTAINING LEVEL POSITION OF I BEAMS
PRESSURE CELL LEADS
SOIL CONTAINER
CONCRETE BASE
WEIGHING TABLE OF TESTING MACHINE

Figure 7.—Plan of Loading Set-Up for Soil Container Tests.

preciable errors in the pressure indications.

Also, the presence of a rim might affect in another way the accuracy with which the cell will indicate pressure because the soil when loaded will tend to flow around the foreign object and, under these circumstances, the pressure intensity on the object would not be uniform from edge to center. If such soil movement tends to form a concentration of pressure around the perimeter of the object, the presence of a rim on the pressure cell should serve to reduce correspondingly the pressure intensity that is indicated by the cell, and this indicated pressure intensity will be less than the average applied over the entire face of the cell. If, on the other hand, there is a concentration of pressure in the central area of the cell face, as might occur with granular material, the pressure intensity indicated by the cell may be higher than the average for the entire cell face.

Earlier in the report mention was made of certain tests in which the pressure cells were installed in a container that was later filled with soil to which load was applied. These tests were designed primarily to develop information concerning the effect on pressure indication of some of the factors just discussed. Figure 7 shows how the equipment was arranged on the table extension of the testing machine that was used to apply known forces on the soil surface.

The soil container consisted of a steel hoop or cylinder 30 inches in diameter and 8 inches high placed on a concrete base which, in turn, was supported on the table of the testing machine. The cells were arranged in the bottom of the container in the pattern shown in figure 8. Loads were applied to the soil with a concrete bearing block that covered the soil surface. Three soils varying in character from a heavy plastic moist clay to a cohesionless dry sand were used in the study. The gradings and soil constants of the materials are listed in table 3. The loam soil was used both in a dry and in a moist condition.

It is recognized that in these tests with the container, the soil through which the pressure was transmitted to the cells was restrained from lateral movement and was thus forced to behave somewhat differently than it would under normal field conditions.

The test procedure was briefly as follows: With the pressure-sensitive area of the cells flush with the surface of the base section, the container was, as a general rule, filled twice in succession with each of the four soils; the load was applied to the bearing block in suitable increments and the indicated pressures were recorded. Then, to cells 2, 3, 5, and 7 the ¾-inch piston extensions were attached and the tests repeated. This was followed by similar tests in which the rim or ring sections were added one at a time to cells 2, 3, and 5.

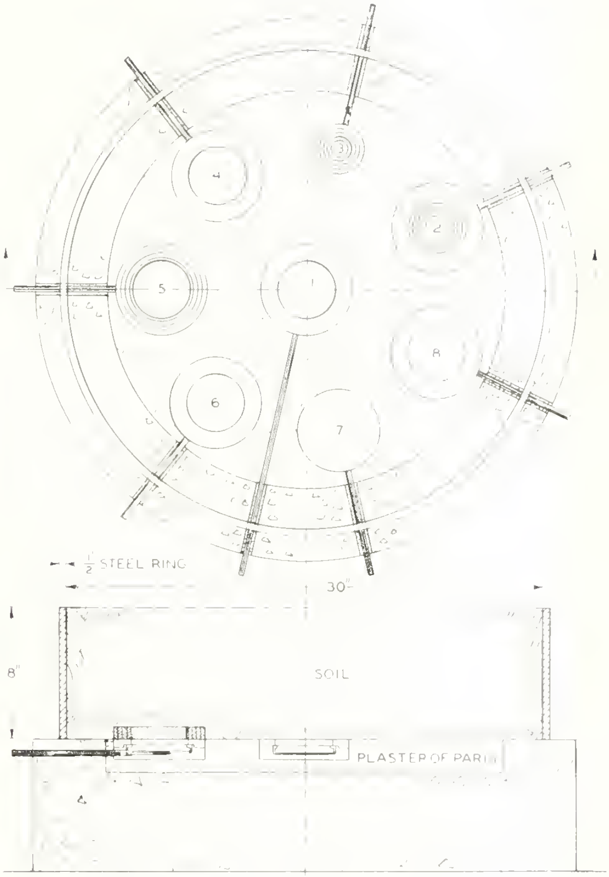

Figure 8.—Lay-out of Cells in the Test Container.

Throughout these tests the pressure-sensitive face of standard cells 1, 4, 6, and 8 remained flush with the base of the container. In the case of cell 7 the ¾-inch-thick piston extension was left in place.

Table 3.—*Gradings and soil constants of materials tested*

MECHANICAL ANALYSIS

Material	Particles larger than 2.0 mm.	Particles smaller than 2 mm. (percent by weight)					
		Coarse sand, 2.0 to 0.25 mm.	Fine sand, 0.25 to 0.05 mm.	Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Colloids, smaller than 0.001 mm.	Passing No. 40 sieve
Sand	0	75	22	1	2	-----	56
Loam	0	14	34	31	21	8	94
Clay	0	2	3	24	71	49	99

PHYSICAL TEST CONSTANTS OF MATERIAL PASSING NO. 40 SIEVE

Material	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
Loam	24	6	15	1.9	18	18
Clay	78	51	15	1.9	43	29

The moisture contents of the damp loam and moist clay soil were held practically constant throughout the tests, being about 15 and 30 percent, respectively. With the exception of the moist clay, which was hand

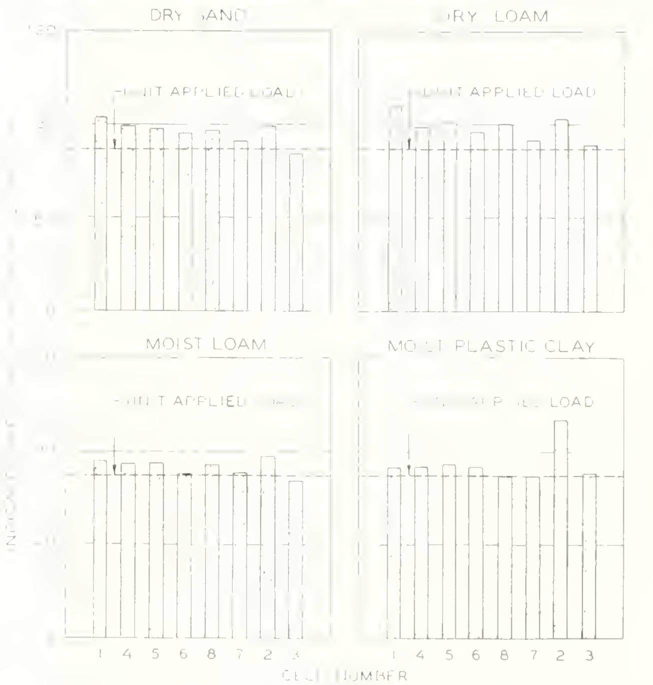

Figure 9.—Variation in Pressure Recorded by the Cells When Flush With the Base of Container.

tamped in place in order to eliminate large void spaces in the final specimen, the soils were placed in the container without artificial compaction. After test, the compacted specimens of the moist and dry loam soil were broken up by screening through a ¼-inch mesh sieve preparatory to being used over again.

With this method of test the effect of size of the pressure-recording device, both with and without encompassing rim sections, could be studied. In contrast to the tests described earlier in the report, the use of the container permitted the soil to be placed or molded against the pressure-sensitive face of the cells. Thus the possibility of nonuniform intimacy of contact between pressure and rim sections was removed.

When these tests were planned it was thought that the intensity of the pressure transmitted through the soil to the base section, at points radially equidistant from the center of the container, would be uniform or at least would not vary to such an extent that the performance of one cell could not be compared directly with another. However, it was found that when the cells were installed flush with the base, differences of appreciable magnitude existed in the pressures indicated by the various cells. The data obtained in the tests with the cell faces flush with the base are given in figure 9. Although it was noted that the differences were greatest between cells of different size and that they were less when plastic soil was used than they were when granular types of material were employed, still no relations could be developed that would permit the direct comparison of the several individual cells.

It was decided, therefore, to use the data obtained with each cell in these tests as a basis of comparison with data obtained with the same cell in future tests in which the individual cells had been altered in various ways. For example, the cell face could be extended into the soil various distances and the effect of varying amounts of rim area could be studied, always comparing individual cell data with those obtained with the same cell set flush with the surface of the base.

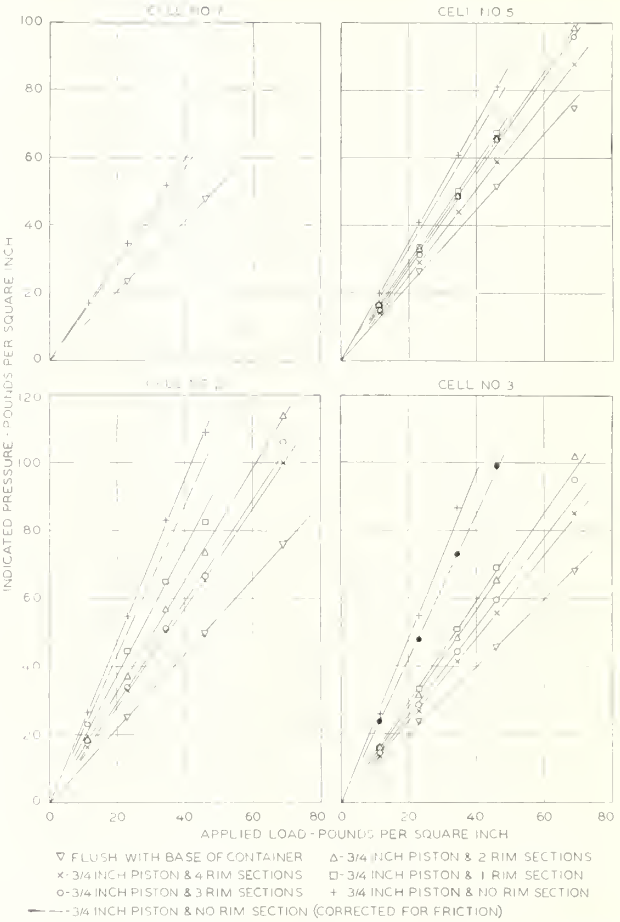

Figure 10.—Load-Pressure Relationships for the Moist Loam Test Soil.

PERFORMANCE OF RIMLESS PRESSURE CELLS DEPENDS UPON THEIR SIZE AND THICKNESS

In general, the pressures indicated by the cells were found to be directly proportional to the applied load. This is evident in the typical load-pressure relationships shown in figure 10 for the moist loam soil. These data show the pressures indicated by cells 2, 3, 5, and 7 for the following physical conditions: (1) Zero thickness, or with the pressure face flush with the base section; (2) ¾-inch thickness, or with the piston extensions attached; and for the ¾-inch thickness plus (3) one rim section, (4) two rim sections, (5) three rim sections, and (6) four rim sections. Each plotted value represents the average of two tests.

In these and subsequent figures and in the test where the term "cell thickness" is used, this dimension refers to the amount of the extension of the cell piston or the cell rim into the soil in the container and does not refer to the dimension of that part of the cell that is embedded in the plaster of paris.

In connection with figure 10, attention is called to the relationship shown by a broken line for each of the cells. This represents a corrected value in which the element of friction on the lateral surface of the piston extension was eliminated. The correction was arrived at by test data obtained with cell 3, in which duplicate tests were made with and without a very thin metal sleeve surrounding the piston extension. It was considered that this sleeve served to eliminate friction between the lateral surface of the piston and the soil.

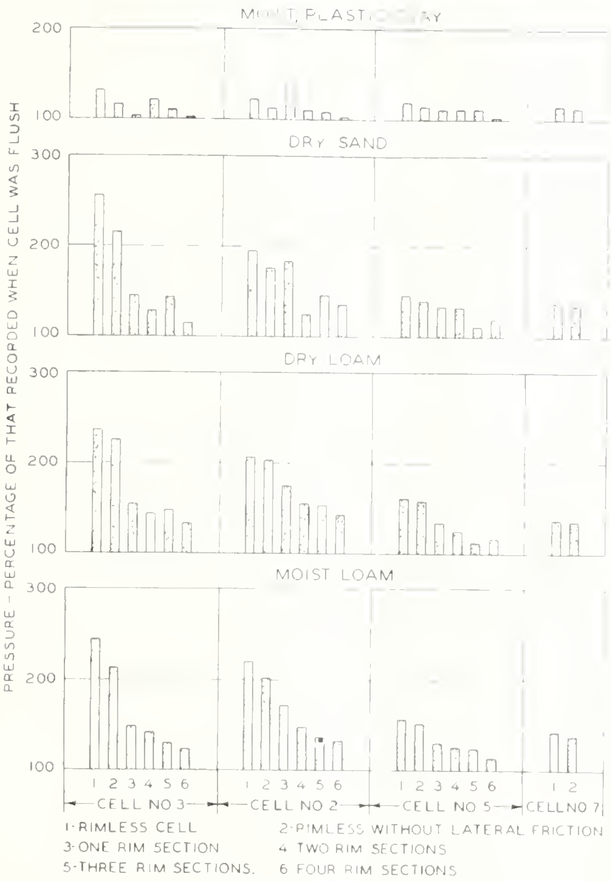

Figure 11.—Performance of Cells With and Without Rim Sections.

Thus the increase in pressure due to cell thickness alone was determined. Assuming that the intensity of this lateral frictional force would be unaffected by the diameter of the cell, the values of pressure that would have been indicated by the other three cells, had not friction been present, were computed.

From such data for the four test soils, it was possible to express on a percentage basis the pressure intensity indicated by the cells when they extended up into the soil, in terms of that recorded when the pressure area was flush with the base of the container. The relationships are shown in figure 11.

With the exception of the test data obtained with the moist plastic clay soil, the trend of the data is very consistent and strongly indicates: First, that as the size of a rimless-type pressure cell is decreased, the pressure intensity that the cell will indicate, in soils that displace vertically above and around them, may be expected to increase to a marked degree; and second, that the presence of a rigid rim section around the pressure-sensitive area tends to compensate for the effect of cell thickness.

The behavior of the moist plastic clay in these tests was quite different from that of the other three soils. Its moisture content was such that apparently the material behaved nearly as a fluid with essentially the same pressure intensity present throughout the confined mass of soil. It was observed in all of the tests with this soil that some of the material, in contrast to the other soils, actually flowed out through the ⅛-inch clearance space between the bearing block and the walls of the container.

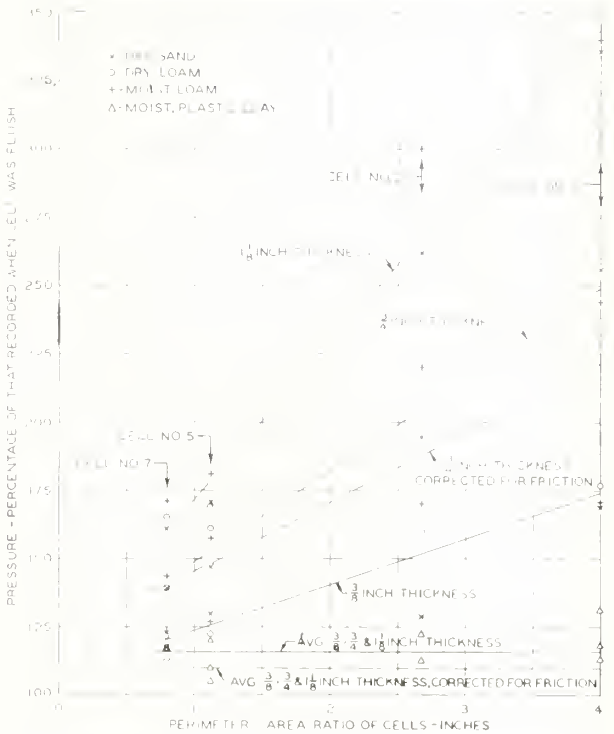

Figure 12. Effect of Size and Thickness of Cells of the Rimless Type.

Figure 12 shows a portion of the same data plotted so as to indicate the effects of the physical dimension of cells of the rimless type on the manner in which they performed under the described conditions of test. In this graph the pressure intensity, expressed as a percentage of that obtained with the cells set flush, is plotted against the perimeter-area ratio of the recording units, for cell thicknesses or piston extensions of ⅜, ¾, and 1⅛ inches, respectively. It was considered advisable to include more than one thickness of cell unit in order properly to evaluate this factor. The relationships shown are considered indicative rather than absolute. However, the indications are believed to be significant.

RIM SECTIONS ON RIGID PRESSURE CELLS COMPENSATE FOR EFFECT OF CELL THICKNESS

It is evident that for the dry sand, dry loam, and moist loam the physical dimensions of the cells, as measured by the thickness and by the perimeter-area ratio, are quite directly related to the accuracy with which they will indicate pressure intensities. In the case of the plastic clay, on the contrary, the physical dimensions of the cells apparently do not affect the accuracy of the pressure indication as is shown by the horizontal line which averages the test values obtained with the plastic clay. It is to be noted that when this value of 117 percent is corrected for lateral friction, it remains somewhat greater than the pressure (100 percent) indicated by the cells when flush with the base of the container. It may be assumed that for this soil the increased pressure intensity indicated by the cells, with the piston extensions in place, was largely due to lateral friction.

The fact that tests with moist loam, dry loam, and dry sand soil gave similar values was not entirely unexpected. In these tests the body of soil as a whole

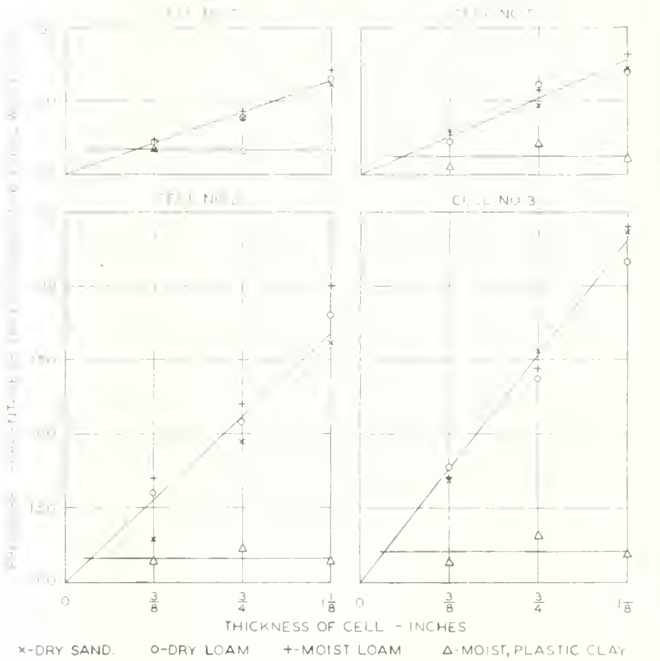

Figure 13.—Effect of Thickness of Cells of the Rimless Type.

could move or displace only in the vertical direction. Had the tests been made in a way such that the material could behave in a natural manner, i. e., displace laterally, the cells in dry sand might have behaved somewhat differently than in the loam soil. It is known that the resistance offered by granular soils to the displacement of loaded bodies varies more or less directly with the size of the loaded area, the displacement under the same unit load decreasing as the size of the area is increased. From this, one would expect that, under normal conditions, the pressure recorded by cells in granular soils would increase as the pressure-sensitive area of the cell was increased, which is contrary to the relation found by the tests made in the rigid container.

Some further evidence indicating that the restraining effect of the container was responsible was obtained from a limited series of tests made in a container whose side walls were not strictly rigid. With this type of container when dry sand was used, the pressure indicated by cells of ¾-inch thickness decreased as their areas were decreased. In fact, for the smallest cell, 1 inch in diameter, the pressure recorded was less when the cell thickness was ¾ inch than when the pressure-sensitive area was flush with the base of the container.

In spite of the fact that the tests were made in a container that restrained the soil body from lateral movement, it is believed that the data shown in figure 12 clearly indicate that, unless a soil is so plastic that it behaves essentially as a fluid, the accuracy of cells of the general design studied may be affected to an important degree by both the thickness and the diameter.

It was remarked earlier in the report that the pressure intensities that rigid cells might indicate when embedded in and surrounded by soil might possibly vary in a manner similar to that which obtains under rigid bearing areas of different sizes. The data obtained in these experiments, as well as data obtained by Housel [4] and other investigators, indicate that this relationship exists.

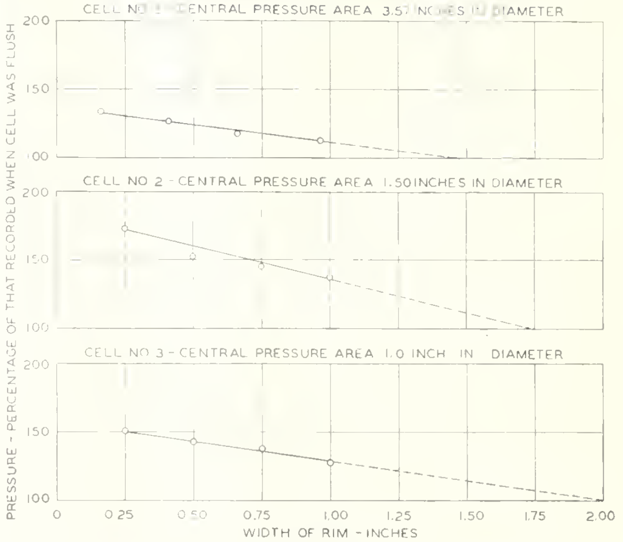

Figure 14.—Effect of Width of Rim for Cells of Different Size.

Referring again to the data given in figure 12, it is of interest to note that the slope of the pressure intensity-perimeter area ratio curves increases as the thickness of the cell unit increases. In other words, as the cell thickness is increased, the differential in soil movement is greater and the effect of diameter (as measured by the perimeter-area ratio) becomes more pronounced.

In his tests with rigid bearing areas, Housel [4] found that as the displacement of the bearing plate was increased, the slope of the pressure intensity-perimeter area ratio curves increased. It is believed that the same soil reaction is responsible in both experiments.

In figure 13, cell thickness is plotted as a direct function of the pressure indicated by each of the four cells for each of the soils tested. It is evident that, unless the soil is so plastic as to behave as a fluid, the indicated pressure intensity varies directly with the thickness of the cell. This effect of cell thickness is a very important one since for practical reasons the cell must have thickness and only in special cases can it be installed so that this thickness does not exert an influence.

The result of increasing the effective rim width of the three cells of variable size, but of a constant ¾-inch thickness, is shown in figure 11. These data show that as the width of rim was increased the pressure intensity indicated by the cell approached that indicated by the same cell when set flush with the base; in other words, that which would be shown by a cell of negligible thickness. This suggests the possibility of determining the width of rim necessary to compensate for the errors in pressure indication that are caused by the cell thickness of ¾ inch.

In figure 14 the data for each of the three diameters of cell (1.0, 1.5 and 3.57 inches, respectively) are arranged for this purpose by averaging the values plotted in figure 11 for the dry sand, dry loam, and moist loam soils for the ¾-inch thickness corrected for side friction. Figure 14 indicates: First, that it is possible to adjust the rim width to compensate for cell thickness in cells of this type; second, that the rim width required for such compensation increases as the over-all cell diameter is decreased; and third, that for cells of this thickness the rim width required is appreciable.

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[4] A Practical Method for the Selection of Foundations Based on Fundamental Research in Soil Mechanics. Department of Engineering Research, University of Michigan, bul. 13, 1929.

SUMMARY

In appraising the significance and utility of the knowledge obtained in this study, it should be remembered that the tests having to do with the influence of the dimensional features on cell performance were made with soil that as a body was not free to move laterally and that was of rather limited depth. How the cells might have performed in the same soils had there been no planes or barriers of discontinuity was not indicated by these tests, but it is probable that some differences would have been found.

The question is intimately related with the state of initial density of the material. Increasing the density of a soil by artificial compaction will reduce its movement under load, although some movement is to be anticipated in even the densest soil materials. To indicate the true pressure developed in a soil by an external force, a pressure-measuring device, particularly a cell of the rimless type, must either possess the same load-displacement characteristics as the soil itself or it must have no tangible thickness. In this connection it appears reasonable to assume that a slight amount of differential movement around a pressure cell in a compacted soil would impose as much additional pressure on the cell as would a relatively large movement in an uncompacted soil.

It seems impracticable to attempt to design pressure-indicating cells that will display the same load-displacement characteristics as the soil because these are neither constant nor known. The design of a cell that has no tangible thickness likewise presents practical difficulties. It appears that the problem of measuring pressures can best be approached by systematic study of the design features of cells that satisfy practical requirements as to dimensions and materials to the end that these may be used with a full knowledge of their accuracy and other characteristics. It is believed that this exploratory study of the performance of cells of a single type, limited as it is, has served a useful purpose and has produced significant results.

It has been shown that when cells equipped with a rigid rim are placed on a soil, even with the greatest care, it is difficult to obtain the same initial intimacy of contact between the soil and the active and inactive areas of the soil face. As a result there is apt to be a rather wide variation in the pressure intensities indicated by identical cells installed in the same way on the same soil medium.

The tests indicate further that when cells of this type are used, the actual magnitude of the pressure intensity that the cell may indicate depends upon the amount of plasticity possessed by the soil. Where the soil used is so plastic as to act like a fluid under pressure, the cells give an accurate indication of pressure intensity.

The accuracy of the indication of pressure intensity given by cells of this type is apparently affected to an important degree by the physical dimensions and by the external design features such, for example, as the relation between the size of the pressure-sensitive area and that of the total facial area exposed to pressure.

It appears possible to design the cell so that, for given test conditions, the error in pressure intensity indication caused by the cell thickness can be compensated for by suitably proportioning the active and inactive areas of the cell face.

As stated at the beginning, this report describes the results of certain exploratory tests. The data obtained are not offered as conclusive. They do, however, point to the importance of a knowledge of the performance of pressure-measuring equipment if dependable data are to be obtained.

(Continued from page 234)

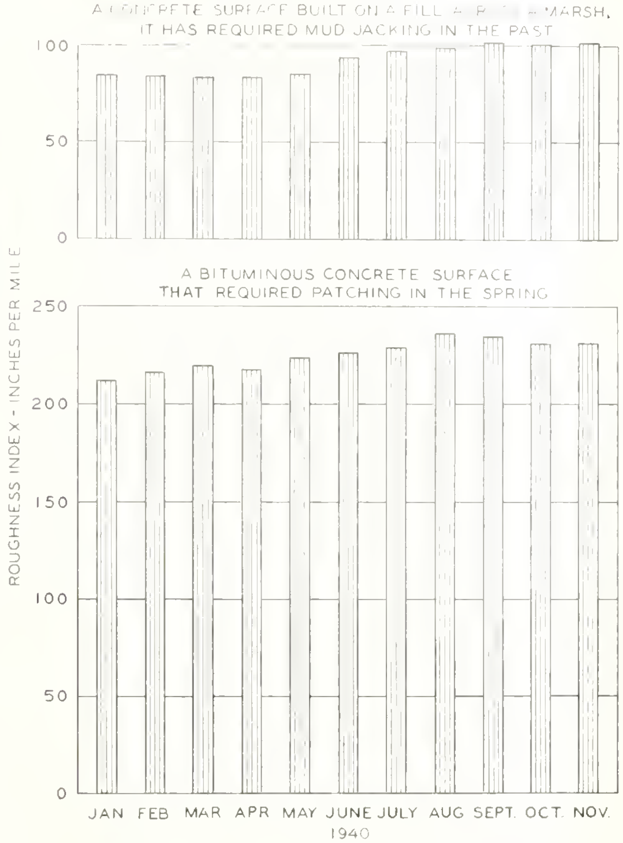

FIGURE 18.—PROGRESSIVE CHANGE IN ROUGHNESS OF TWO ROAD SURFACES WITH TIME.

SUMMARY

The equipment that has been described is not perfect and it may be that some changes in the apparatus or in the technique of field testing may be found desirable at some future time. It is believed, however, that the underlying principle as a means for indicating the relative roughness of road surfaces is sound and that the present equipment is superior to that developed earlier in two important respects. First, the introduction of a special vehicle has removed the uncertainties of vehicle operation that were always present when an automobile was a component part of the measuring apparatus; and second, the entire equipment is so designed that it can be exactly duplicated and, to this extent, the equipment is standardizable. Many hundred miles of operation without mechanical failure indicate adequate mechanical design. The equipment is stable in operation and has shown remarkable consistency of performance. Finally, it is easy to use and the data are obtained rapidly with it.

It would appear that there are several fields of usefulness for equipment of this kind. If the apparatus is constant in its performance, it permits of periodic surveys of surface condition that will give quantitative data on the roughness of the various pavements in a highway system. For example, figure 18 shows how the surfaces of two pavements compared month by month over a period of 11 months. The concrete pavement rests on a fill that is not completely stable and the bituminous pavement is one that requires patching from time to time. Data such as these show the need for and result of surface maintenance operations. With means for evaluating surface roughness available, the interest of construction and maintenance crews can be aroused and a desirable competitive spirit stimulated.

REPORT OUTLINING NATION'S HIGHWAY NEEDS STILL AVAILABLE

Copies of the report "Toll Roads and Free Roads" are available from the Public Roads Administration of the Federal Works Agency. This publication is divided into two parts as follows:

1. It reports the findings of an investigation of a proposal to build six toll highways, three spanning the country from east to west, and three from north to south.
2. It presents a master plan for future highway development, based on factual information obtained in highway planning surveys.

Initial distribution of "Toll Roads and Free Roads" has already been made. However, an ample supply of this particular publication is still available for free distribution to highway engineers, libraries, colleges, and others. Requests for "Toll Roads and Free Roads" should be addressed to the Public Roads Administration, Washington, D. C.

SIMILAR EQUATIONS DERIVED BY INDEPENDENT WORKERS

In the article "Soil Displacement Under a Loaded Circular Area," by L. A. Palmer and E. S. Barber appearing in the December 1940 issue of PUBLIC ROADS, reference to an earlier article by the late Dr. John H. Griffith of Iowa State College was unfortunately not made. The authors derived their equation 5 without knowledge that Dr. Griffith had presented essentially the same expression as his equation 3 on page 32 of Bulletin 101, volume 30, No. 1, Iowa Engineering Experiment Station, Iowa State College, under the title "Physical Properties of Earths." Equation 5 in the report by Palmer and Barber is

$$V=\frac{p}{C}\left[(2-2\mu^2)\ (a^2+z^2)^{1/2}-\frac{(1+\mu)z^2}{(a^2+z^2)^{1/2}}+(\mu+2\mu^2-1)z\right]\text{------}(5)$$

where V is the vertical displacement of an earth particle at any depth, z, on the axis of symmetry under a uniformly loaded circular area at the ground surface; μ is Poisson's ratio; a is the radius of the loaded circular area; p the unit load; and C is the modulus of deformation obtainable from plotted stabilometer test data.

In Dr. Griffith's equation 3, a modulus of elasticity, E, was used in place of the C in the above equation. Dr. Griffith did not indicate values for μ or E nor did he mention any possible laboratory means of determining these values. Aside from this difference, his expression is the same as the above equation 5.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1941

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			*BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,858,664	$ 1,421,234	95.1	$ 5,233,779	$ 2,603,570	189.8	$ 977,800	$ 486,350	32.9	$ 3,818,639
Arizona	1,054,331	739,348	47.7	1,968,869	1,313,364	83.1	428,157	266,227	9.7	1,759,129
Arkansas	4,865,206	2,231,305	124.5	1,276,678	636,963	50.9	296,709	147,722	8.0	1,738,999
California	5,812,677	2,948,188	110.0	8,206,550	4,363,159	127.1	1,835,549	942,990	35.5	4,528,248
Colorado	2,172,320	1,183,569	188.7	2,001,971	1,147,250	81.0	798,749	450,175	89.6	3,385,286
Connecticut	1,936,006	938,685	16.2	1,187,936	580,855	11.3	699,855	338,145	7.5	1,313,013
Delaware	1,085,086	541,827	24.0	1,107,587	527,071	8.5				1,530,883
Florida	2,512,745	1,248,025	62.5	1,830,844	923,710	59.4	618,194	309,097	25.6	3,687,669
Georgia	3,033,370	1,508,949	184.7	7,077,312	3,539,156	277.3	1,730,704	865,352	79.9	6,926,356
Idaho	1,529,343	916,941	151.0	1,040,215	640,970	56.9	505,148	311,354	12.5	2,296,138
Illinois	6,435,116	3,184,386	146.0	6,775,176	3,387,423	132.3	2,746,200	1,373,100	90.0	5,364,167
Indiana	5,063,556	2,493,378	117.6	6,438,874	3,113,291	90.0	2,072,267	962,200	42.3	2,689,881
Iowa	5,486,851	2,563,365	188.6	3,861,243	1,731,009	119.4	1,153,219	470,421	66.9	2,546,581
Kansas	3,651,486	1,801,155	301.2	5,761,217	2,899,371	346.6	2,811,870	1,393,916	167.8	5,237,218
Kentucky	2,756,915	1,373,701	85.2	3,012,306	1,511,833	81.1	931,557	465,779	37.3	4,329,017
Louisiana	1,183,855	586,339	16.0	12,581,978	3,339,976	63.2	969,534	477,079	34.9	4,251,218
Maine	1,319,269	641,968	29.1	734,950	381,115	20.5	841,245	420,622	1.1	1,067,035
Maryland	1,019,901	505,992	24.2	3,121,923	1,556,361	27.2	494,303	247,151	5.9	2,032,567
Massachusetts	1,669,600	831,974	19.9	2,360,660	1,179,910	16.1	61,470	30,705	.4	4,307,283
Michigan	6,132,119	2,904,437	215.1	8,131,410	4,053,105	197.0	775,610	387,805	24.3	3,051,351
Minnesota	5,941,448	2,902,882	448.1	4,305,316	2,149,276	244.1	1,642,752	820,446	69.9	5,138,495
Mississippi	1,916,346	777,748	94.7	7,202,034	3,358,161	353.3	1,160,300	557,350	85.1	2,785,895
Missouri	3,412,475	1,690,301	169.0	7,823,951	3,649,255	194.0	4,325,104	1,967,383	124.7	5,293,489
Montana	4,086,764	2,313,548	283.3	2,202,220	1,242,712	116.7	692,624	390,848	40.7	4,944,690
Nebraska	4,643,695	2,217,388	548.3	3,838,524	1,946,325	445.3	1,861,780	930,889	197.5	3,764,135
Nevada	1,545,941	1,322,120	79.6	1,038,047	903,974	43.9	381,593	293,814	23.8	1,750,673
New Hampshire	1,445,362	707,199	36.4	419,795	209,610	9.0				1,401,951
New Jersey	1,525,860	751,438	9.4	5,681,620	2,840,810	39.3	1,882,890	941,365	15.2	1,754,615
New Mexico	2,043,679	1,243,663	172.8	1,521,249	928,805	67.0	373,787	225,318	10.9	2,336,740
New York	11,443,080	5,610,806	198.8	11,258,818	5,601,260	137.8	692,127	275,809	12.0	4,993,656
North Carolina	4,326,497	2,161,479	232.2	4,492,562	2,231,595	184.2	1,166,250	582,630	58.9	3,193,427
North Dakota	1,951,966	1,049,687	191.2	2,532,510	1,426,041	197.4	2,549,484	1,295,325	217.0	4,645,532
Ohio	4,729,002	2,363,683	61.6	11,961,842	5,956,727	100.8	8,024,590	3,933,132	69.3	4,778,415
Oklahoma	2,403,776	1,274,725	112.6	2,825,034	1,454,589	82.2	1,833,026	954,127	77.2	5,694,541
Oregon	3,099,420	1,852,951	152.6	2,409,715	1,289,526	44.5	875,385	460,203	36.8	2,068,943
Pennsylvania	5,703,638	2,821,970	76.0	13,417,474	6,655,924	109.7	2,377,674	1,145,858	19.1	4,672,511
Rhode Island	1,028,203	511,131	10.2	1,195,010	596,687	10.9	4,760	2,380		1,256,946
South Carolina	1,730,969	835,385	120.6	1,940,197	919,075	133.8	1,979,190	864,790	61.3	2,799,238
South Dakota	3,145,039	1,766,352	530.9	3,897,733	2,458,183	462.9	883,560	509,000	153.7	3,506,320
Tennessee	2,438,142	1,210,196	57.1	3,059,792	1,529,896	108.4	2,349,672	1,174,836	57.5	4,785,322
Texas	7,312,763	3,586,840	463.9	9,387,080	4,638,106	442.5	4,878,879	2,401,940	155.5	9,118,009
Utah	985,953	714,222	73.1	913,421	683,969	39.7	543,279	293,840	13.4	1,672,175
Vermont	1,187,876	589,160	36.6	758,933	379,569	20.1	450,599	225,285	12.9	576,523
Virginia	2,306,123	1,068,350	64.8	3,746,689	1,777,135	70.3	798,797	396,585	10.4	2,717,689
Washington	3,110,393	1,603,560	78.2	3,134,689	1,665,335	33.8	229,841	122,100	2.5	1,832,528
West Virginia	1,989,390	991,108	74.2	3,035,284	1,511,440	63.2	1,374,486	684,810	20.2	1,930,086
Wisconsin	5,220,652	2,552,603	179.1	2,015,484	997,835	88.9	937,327	464,953	23.0	5,384,677
Wyoming	1,793,781	1,100,281	196.2	980,178	619,650	120.0	605,717	387,171	41.7	1,593,691
District of Columbia	533,627	266,813	5.0	602,937	269,909	.8	134,400	67,200	.9	614,339
Hawaii	110,831	51,536	1.7	713,726	377,860	10.2	138,944	69,472	2.5	1,953,237
Puerto Rico	519,644	257,240	10.9	1,370,673	677,010	20.3	179,616	88,630	2.0	1,047,259
TOTALS	155,210,751	78,731,131	6,916.4	203,394,015	100,375,711	6,033.7	56,076,573	32,873,679	2,387.7	165,867,425

* INCLUDES APPORTIONMENT FOR THE FISCAL YEAR 1942

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JANUARY 31, 1941

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			* BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 190,944	$ 95,263	9.4	$ 1,308,357	$ 654,158	60.7				$ 640,736
Arizona	144,270	83,190	17.3	217,025	156,676	7.0				475,609
Arkansas	416,160	179,101	14.8	306,339	152,958	20.6	$ 148,414	$ 73,800	12.2	311,721
California	831,102	451,771	37.3	726,179	493,962	10.2	586,137	287,457	11.6	895,657
Colorado	45,873	25,854	1.6	252,726	141,919	5.3	35,799	20,176	2.9	411,324
Connecticut	370,531	179,413	4.6	105,456	49,907	1.8	194,675	87,585	4.3	186,667
Delaware	128,160	56,367	12.7	46,219	22,675	2.8				350,778
Florida	12,030	6,015		604,373	302,186	14.7	550,715	241,531	5.8	329,157
Georgia	147,586	73,258	18.6	823,559	396,780	64.7	297,409	148,705	32.6	1,381,251
Idaho	153,024	94,195	24.0	166,859	99,425	6.7	72,833	4,293	3.5	324,457
Illinois	1,695,358	827,554	80.6	939,350	454,675	28.2	541,100	245,750	39.0	697,818
Indiana	476,372	229,757	31.0	163,864	87,389	8.1	219,571	109,785	10.0	1,230,526
Iowa	2,272,828	1,077,776	487.0	638,984	303,790	176.9	262,187	113,000	77.9	492,184
Kansas	320,172	159,770	49.0	913,408	459,612	46.8	175,115	87,558	51.8	1,574,625
Kentucky	792,145	268,935	65.5	569,227	138,095	18.2	373,349	106,751	21.6	552,479
Louisiana	98,857	49,429	10.9	199,072	99,481	14.9				717,587
Maine	298,852	142,635	17.0	40,606	20,303	1.5				157,657
Maryland	128,300	64,150	5.5	98,390	49,195	4.5	36,000	18,000	4.3	531,665
Massachusetts	456,347	225,862	10.3	220,045	113,703	3.6				702,518
Michigan	1,443,931	712,804	112.3	597,780	298,890	35.9	537,720	268,860	39.5	720,827
Minnesota	734,528	358,911	103.9	667,878	333,939	102.2	449,535	224,768	45.4	1,156,016
Mississippi	272,962	136,481	12.5	660,552	326,126	37.0	325,700	140,365	14.4	743,870
Missouri	732,486	363,341	96.5	112,644	56,322	10.4	286,710	120,187	27.0	1,142,650
Montana	641,506	362,577	80.3	90,699	51,181	3.7	263,163	145,257	55.2	852,529
Nebraska	493,800	240,550	88.0	648,882	324,218	79.1	62,260	31,130	17.3	545,812
Nevada	196,240	166,314	40.9	175,048	152,379	14.3				238,859
New Hampshire	143,639	68,883	3.4	71,533	34,946	3.6				219,539
New Jersey	319,476	159,633	10.6	322,057	162,940	11.4	75,810	37,905	.2	685,061
New Mexico	94,763	59,142	13.1	634,137	343,277	28.8	92,695	49,568	3.7	326,218
New York	2,027,620	970,482	67.9	1,347,060	673,530	40.0	205,000	62,297	.6	851,636
North Carolina	946,392	471,070	82.2	378,303	191,763	40.1	56,100	24,500	3.1	609,631
North Dakota	42,880	24,583	.3	169,224	90,702	3.6				1,278,769
Ohio	1,656,279	827,346	56.0	1,807,235	902,358	57.5	220,375	110,187	11.3	1,266,827
Oklahoma	667,352	353,621	47.8	216,680	114,450	12.4	231,100	121,589	11.7	1,209,791
Oregon	372,237	205,770	56.4	190,645	84,674	13.4	250,561	126,040	22.9	413,125
Pennsylvania	1,569,327	777,140	55.9	924,097	461,140	17.6	321,936	146,468	6.7	775,751
Rhode Island	245,259	122,207	3.6	90,306	47,016	.9				126,404
South Carolina	569,763	209,926	79.0	350,840	116,990	16.9	361,867	158,700	45.6	307,042
South Dakota				28,926	19,392	9.0				1,543,134
Tennessee	151,200	72,135	8.7	287,466	143,733	10.0				1,235,745
Texas	1,187,474	579,866	176.4	1,264,264	627,228	103.5	187,956	91,650	25.6	1,646,880
Utah	88,404	49,100	9.5	185,785	123,660	22.1	55,085	27,543	2.5	294,647
Vermont	338,432	108,810	13.1	193,984	56,235	7.6				97,105
Virginia	387,164	181,027	24.8	556,520	259,546	19.7				508,533
Washington	469,256	245,605	28.2	286,980	153,439	12.2	128,257	68,700	16.2	403,637
West Virginia	310,127	154,327	16.7	118,300	59,150	4.2	43,400	21,700	2.1	600,264
Wisconsin	331,132	164,511	7.4	658,495	330,305	23.2	497,948	184,628	20.2	863,672
Wyoming	429,615	253,344	42.8	153,530	95,100	9.1				289,633
District of Columbia	120,324	60,162	1.4	2,192	1,096					111,865
Hawaii	275,662	137,588	8.6	2,192	2,192					244,452
Puerto Rico	88,612	43,260	4.3	213,612	104,380	9.7	55,188	27,140	2.1	167,407
TOTALS	26,326,753	12,930,811	2,249.6	21,747,884	10,939,186	1,256.3	8,201,670	3,733,573	650.8	33,441,747

* INCLUDES APPORTIONMENT FOR THE FISCAL YEAR 1942

U. S. GOVERNMENT PRINTING OFFICE 1941

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–20, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JANUARY 31, 1941

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					• BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 28,328	$ 28,328			4	$ 750,688	$ 730,611	7	1		$ 189,293	$ 189,293	2		8	$ 1,129,534
Arizona	198,347	190,981	3		2	179,037	178,688	1			13,255	13,255			2	362,838
Arkansas	651,975	651,441	6	1	8	681,768	677,936	7			341,631	341,466	3		9	473,309
California	445,134	445,134	4		8	1,028,139	847,540	7	1		754,038	754,038	2		13	1,403,058
Colorado						288,868	288,868	1			349,978	349,978	2		3	823,777
Connecticut	622,002	611,366	5	1		166,222	165,415	2								614,980
Delaware	68,080	68,080			13	132,406	132,406	5		3	2,332	2,332			1	560,165
Florida	207,524	203,025	2		2	102,816	102,291	1	1		142,649	142,649			26	1,508,766
Georgia	182,417	182,322	4	1	13	1,083,508	1,083,508	10	6		249,272	249,272	1	2	5	2,169,769
Idaho	259,459	256,029	5		23	27,239	27,239			4	30,422	30,422			10	569,399
Illinois	1,700,569	1,630,832	9	1	42	1,343,252	1,114,152	4	1	90	185,043	176,949	1		31	2,896,334
Indiana	675,332	673,532	3	2	79	611,442	611,442	5		14	451,341	418,441	3	1	8	1,032,980
Iowa	476,834	449,274	4		81	164,783	136,372	2		13	292,461	291,381	3	1	10	1,462,325
Kansas	691,995	691,888	9		15	453,190	452,712	4	1	5	383,372	383,372	10	1	8	1,226,875
Kentucky	574,050	572,380	8		11	638,550	638,550	7		1	377,956	339,556	2		3	605,766
Louisiana	95,496	95,496	1			345,122	291,627	2			632,768	575,037	11		2	1,131,893
Maine	159,759	158,841	1	1	6	132,646	132,646	1	1							398,661
Maryland	180,997	180,993	1		2	485,009	453,216	2	2	7	7,200	7,200			1	973,714
Massachusetts	16,588	16,588		1		342,715	332,292	1			90,040	89,740	1			2,319,006
Michigan	1,157,264	1,122,764	8	1	35	1,431,235	1,431,235	2	5	2	157,807	157,807	2		6	1,125,379
Minnesota	1,131,064	1,122,664	8	2	13	983,368	983,368	9	4	6	149,645	149,645		2	2	1,510,210
Mississippi	260,760	260,760	3	1		598,234	598,234	8		1	86,900	86,900	1		3	803,304
Missouri	1,208,389	1,208,389	6	5	1	1,709,501	1,254,081	5	1		215,040	161,640		2		1,461,573
Montana	427,675	427,675	5		2	88,047	88,046	1			9,155	9,155			3	609,220
Nebraska	424,946	423,276	3	1	10	813,918	813,918	13	1		130,928	130,928	1		13	512,622
Nevada	71,422	71,422	1		19	69,039	69,039	1			72,643	72,643	1		6	156,383
New Hampshire	100,989	100,953	3			149,458	148,638	3		1						429,450
New Jersey	280,886	280,886	2		4	585,098	585,098	3	1		531,926	403,881	2		1	1,173,741
New Mexico	242,979	242,979	2		6	183,821	175,247	3								651,420
New York	1,207,716	1,173,326	7	5		3,037,233	2,986,562	5	15		893,345	853,094	2	2	8	3,562,126
North Carolina	550,228	550,165	8	2	20	644,889	644,649	6	4	5	167,670	167,670		2	29	1,237,284
North Dakota	42[illegible],932	428,538	5		22	385,790	385,790	4		1						592,264
Ohio	814,523	792,761	6		17	2,140,570	2,081,380	9	3		865,625	850,163	5			2,890,830
Oklahoma	519,818	517,444	9	1	33	428,397	424,981	4		1	113,688	113,688			37	2,319,290
Oregon	208,639	117,537	3			335,392	272,981	3			5,790	5,790			2	521,531
Pennsylvania	1,387,269	1,377,793	13	2		2,021,009	2,017,181	14			846,854	846,854	7			3,746,923
Rhode Island	8,222	7,406				206,703	206,703		1							175,628
South Carolina	431,762	431,359	4	3	17	100,017	100,017	1		5	263,196	263,196	2	1	32	1,094,416
South Dakota	129,470	129,470	2	1	4	528,394	527,534	14			144,650	128,700	4		1	1,017,377
Tennessee	244,886	241,169	2	2		166,727	166,727		2		162,270	153,270	3	1		2,005,864
Texas	1,271,717	1,266,908	11		9	1,346,378	1,332,513	14	1		217,310	163,350	2		2	2,744,973
Utah	32,086	31,875			9	100,628	99,886	2		35	73,389	73,389			29	341,000
Vermont	111,265	111,095		1	6	145,570	145,570		1	6						312,120
Virginia	196,544	195,098	2	2	5	679,220	679,000	5	2	2	138,847	138,847	1	1	9	870,779
Washington	266,537	263,586	4	1	6	533,933	532,433	3	4	4	11,529	11,529			4	455,273
West Virginia	9,310	9,310			2	440,042	434,422	5			2,820	2,820			1	1,200,243
Wisconsin	834,596	819,760	6	4	7	452,980	423,971	3		4	143,882	143,668	1		16	1,724,962
Wyoming	1,986	1,982				560,904	560,903	6			4,997	4,997			2	299,298
District of Columbia	56,868	56,868	1			2,193	2,193									275,206
Hawaii	4,810	4,810			1	200,913	200,906	2		1						388,597
Puerto Rico						584,007	579,336	11								582,439
TOTALS	21,259,444	20,896,558	189	42	557	30,611,008	29,350,053	228	59	211	9,902,957	9,448,005	75	16	346	58,754,874

• INCLUDES APPORTIONMENT FOR THE FISCAL YEAR 1942

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ABBREVIATIONS

a.–article(s), report(s)
fn.–footnote
r.–reference(s), referred to
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